CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

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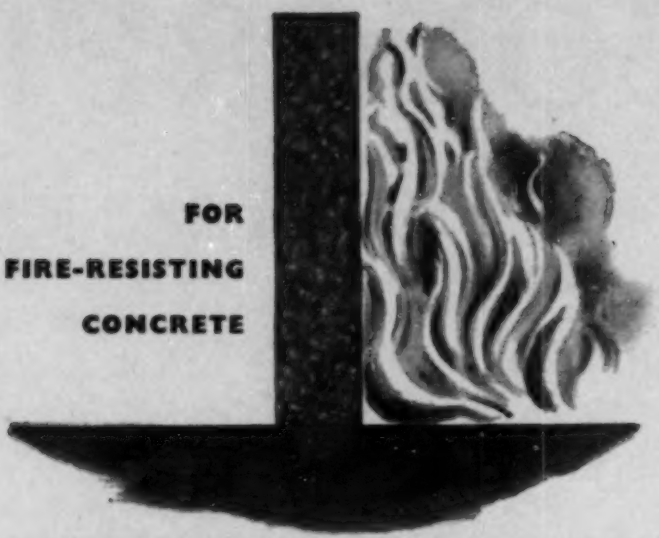

FOR
FIRE-RESISTING
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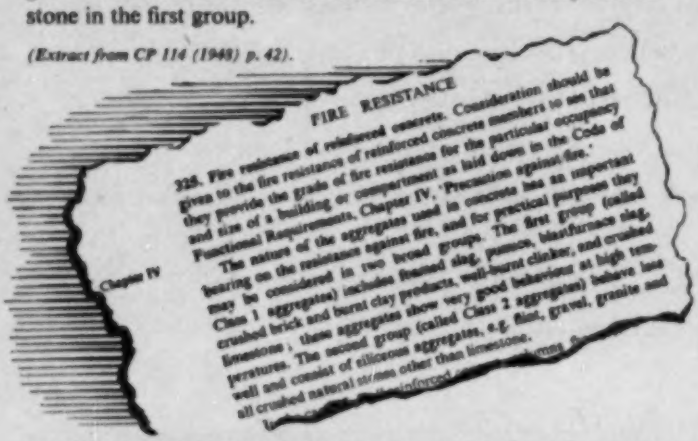
LIMESTONE AGGREGATE

The extract from the British Standard Code of Practice CP 114 (1948) shown below gives clear-cut information on the fire-resistance of reinforced concrete. It divides the nature of aggregates for practical purposes into two broad groups. The first group gives better results than the second. Limestone is the only natural stone in the first group.

(Extract from CP 114 (1948) p. 42).



FIRE RESISTANCE

328. Fire resistance of reinforced concrete. Consideration should be given to the fire resistance of reinforced concrete members to see that they provide the grade of fire resistance for the particular occupancy and size of a building or compartment as laid down in the Code of Functional Requirements, Chapter IV, 'Precautions against fire.'

The nature of the aggregates used in concrete has an important bearing on the resistance against fire, and for practical purposes they may be considered in two broad groups. The first group (called Class 1 aggregates) includes foamed slag, pumice, blastfurnace slag, crushed brick and burnt clay products, well-burnt clinker, and crushed limestone; these aggregates show very good behaviour at high temperatures. The second group (called Class 2 aggregates) behaves less well and consist of siliceous aggregates, e.g. flint, gravel, granite and all crushed natural stones other than limestone.

Chapter IV

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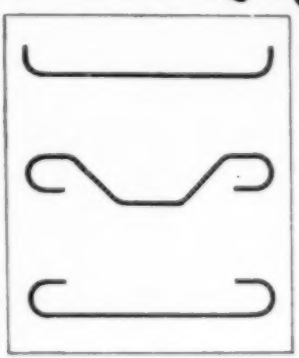

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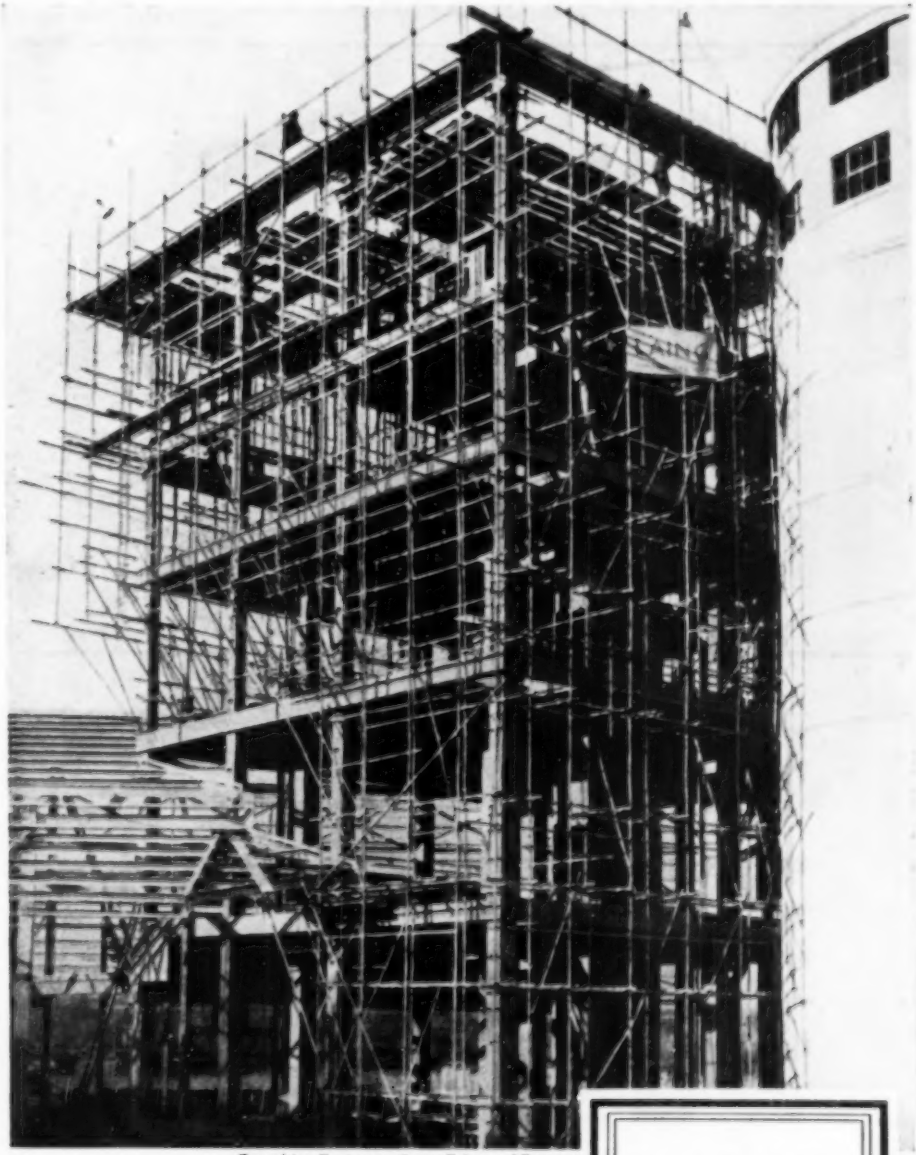

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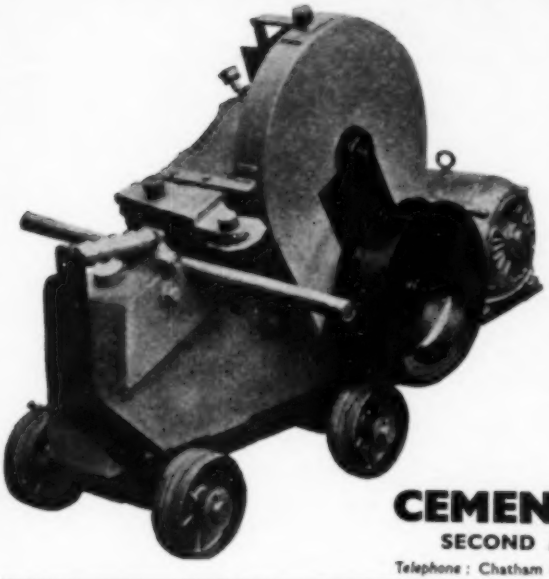

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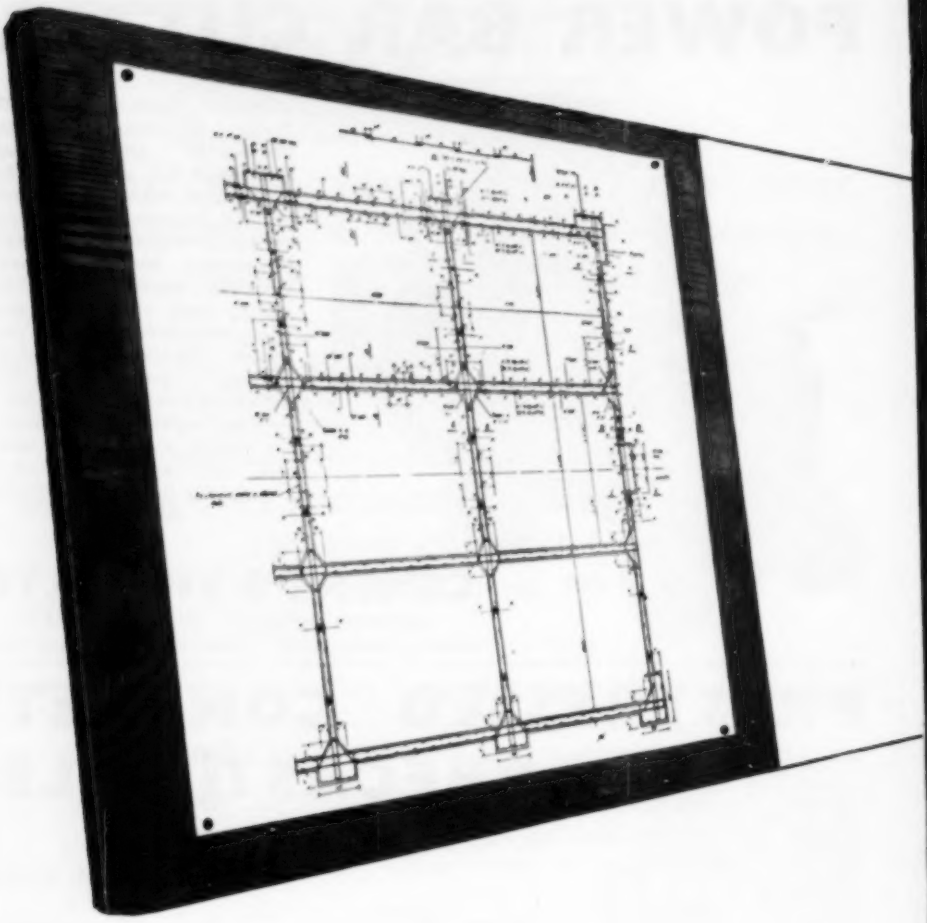

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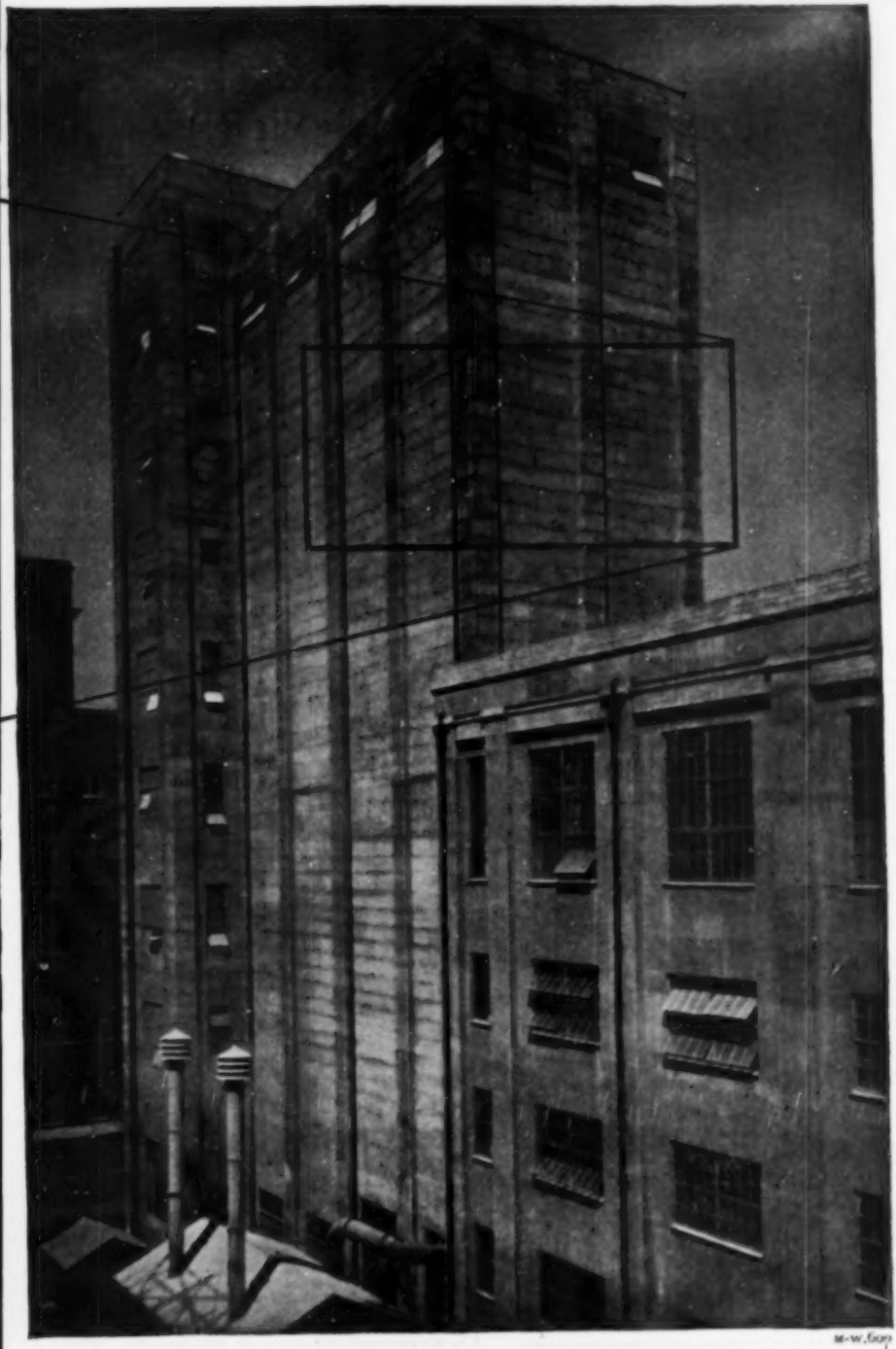

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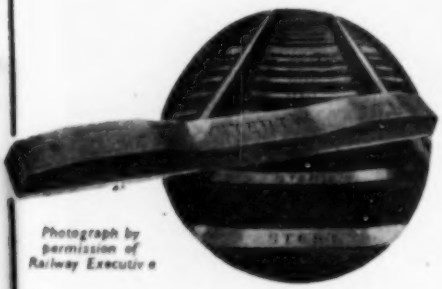

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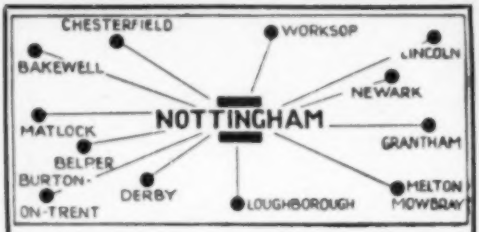

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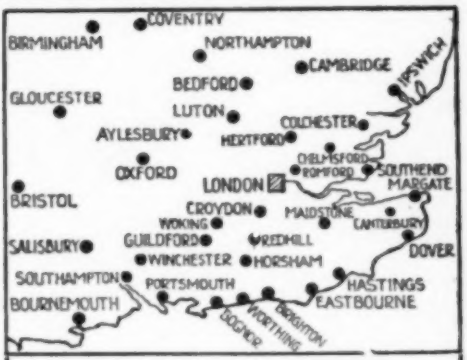

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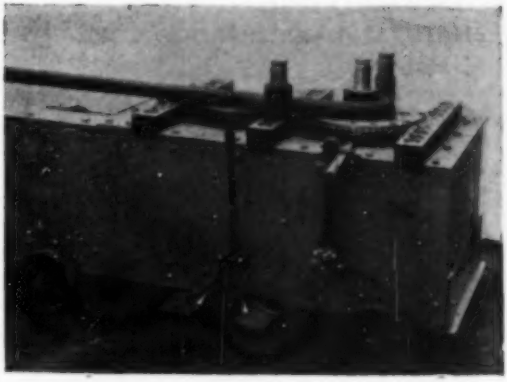

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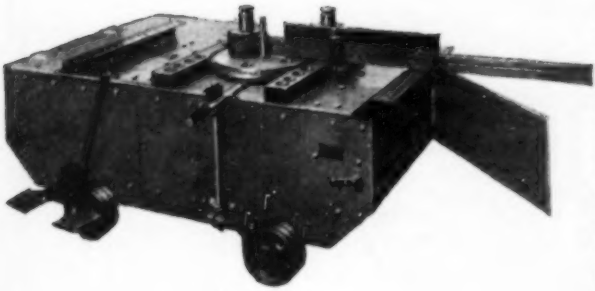

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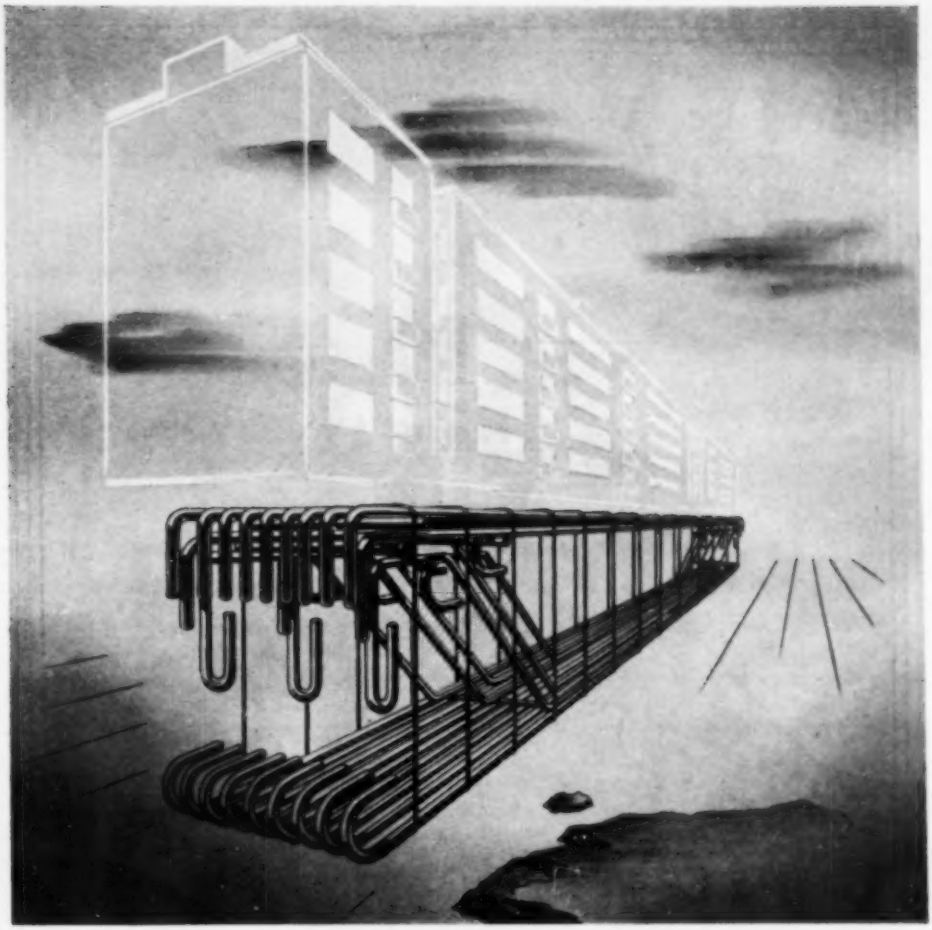

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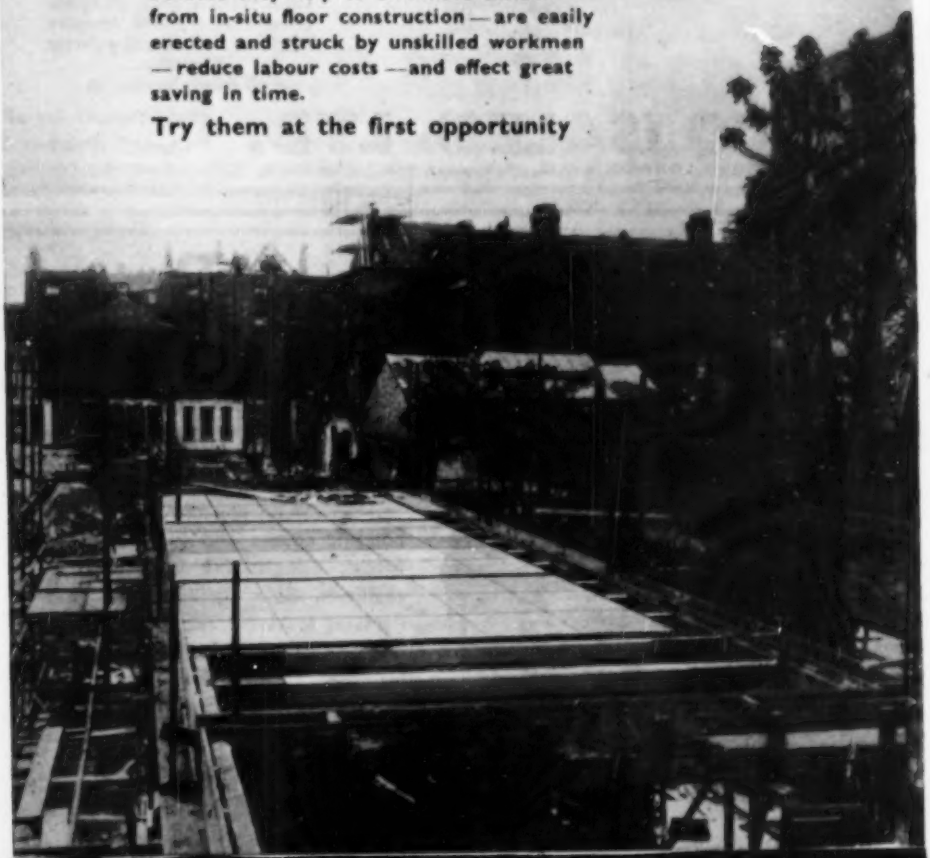

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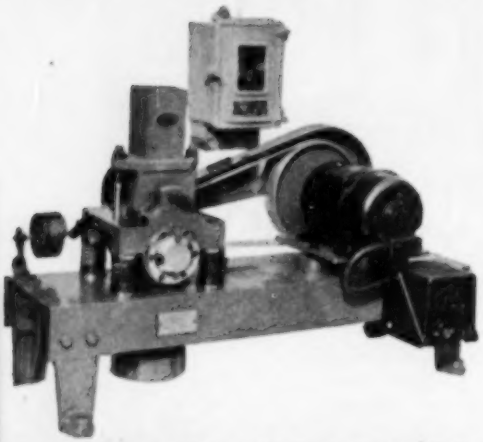

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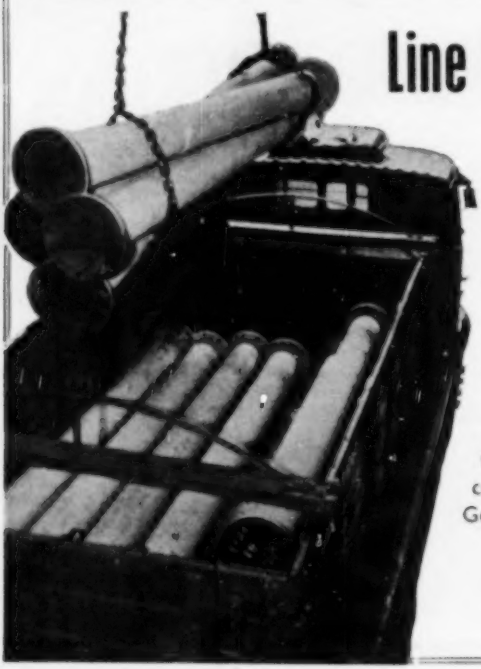

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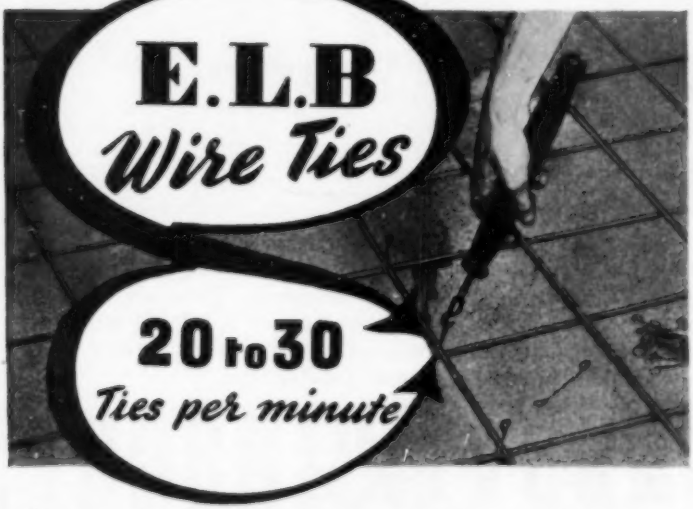

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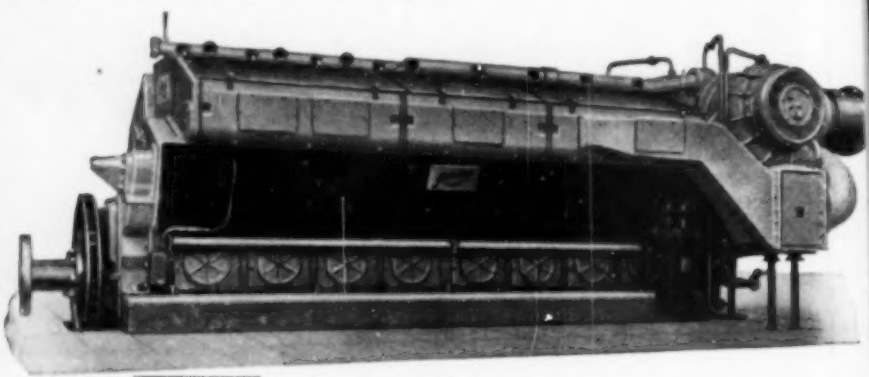

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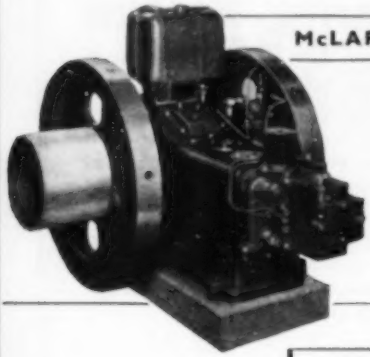

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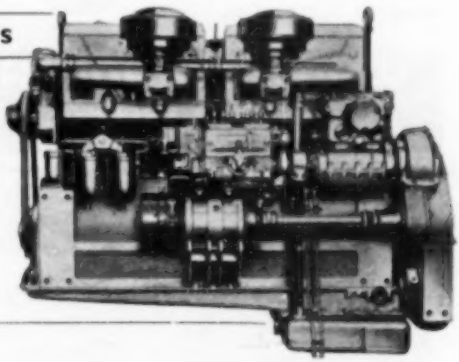

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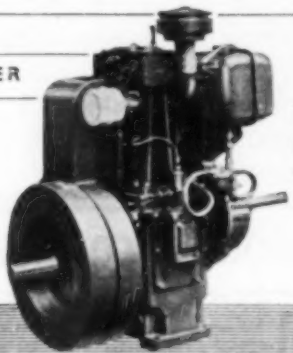

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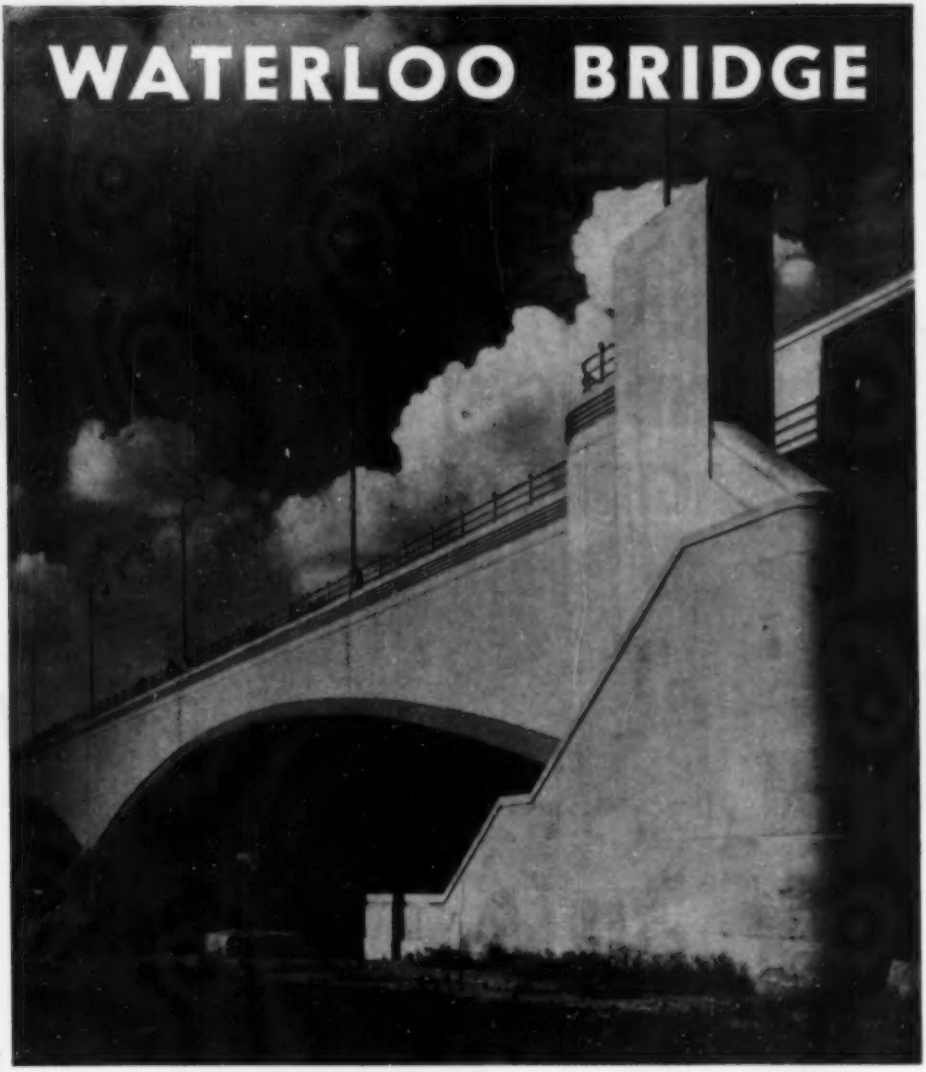

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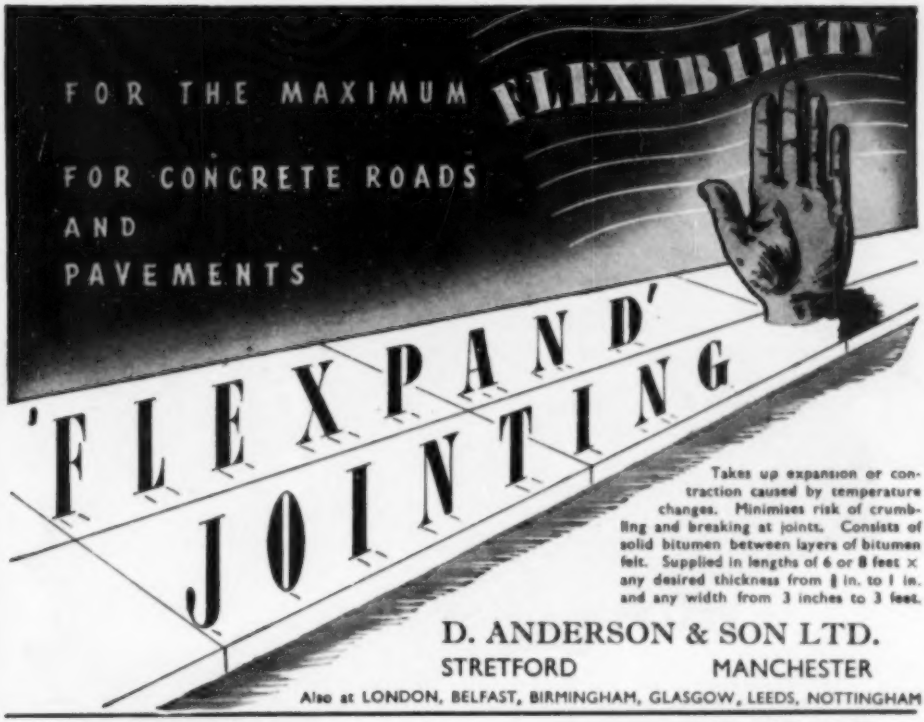

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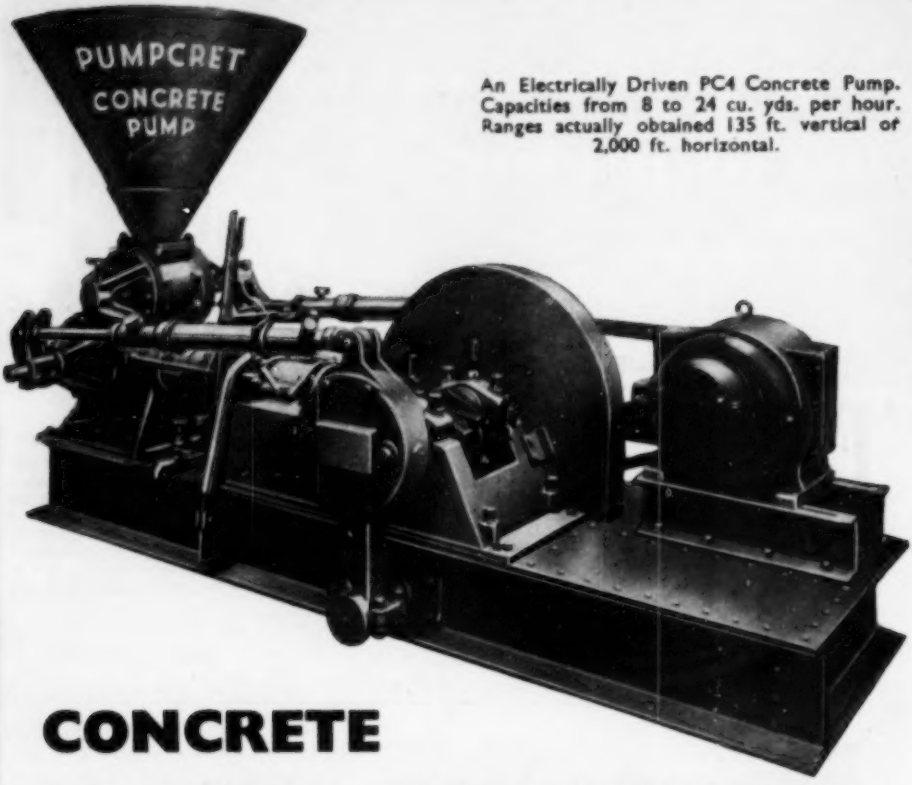

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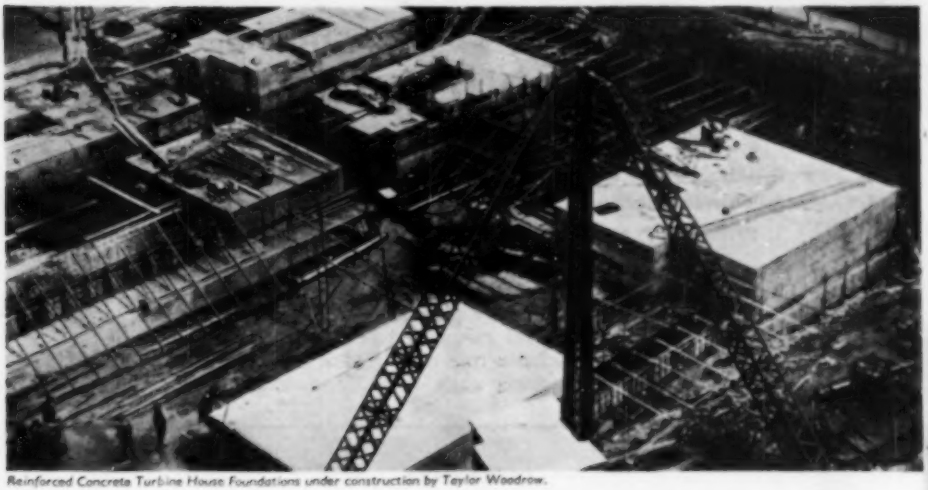

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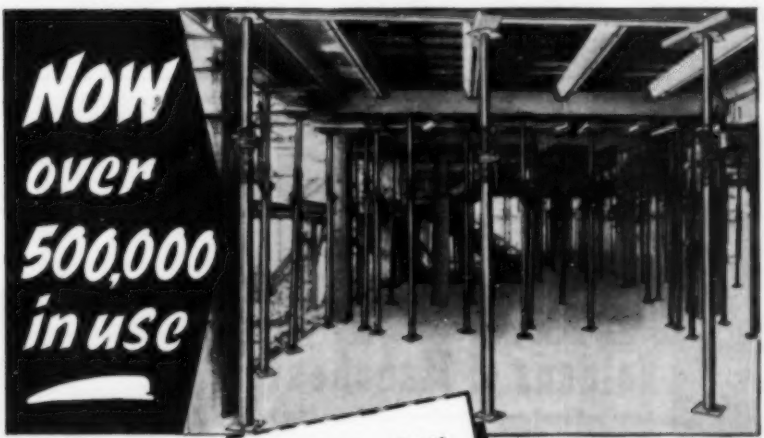

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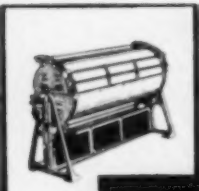

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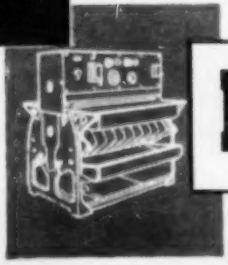

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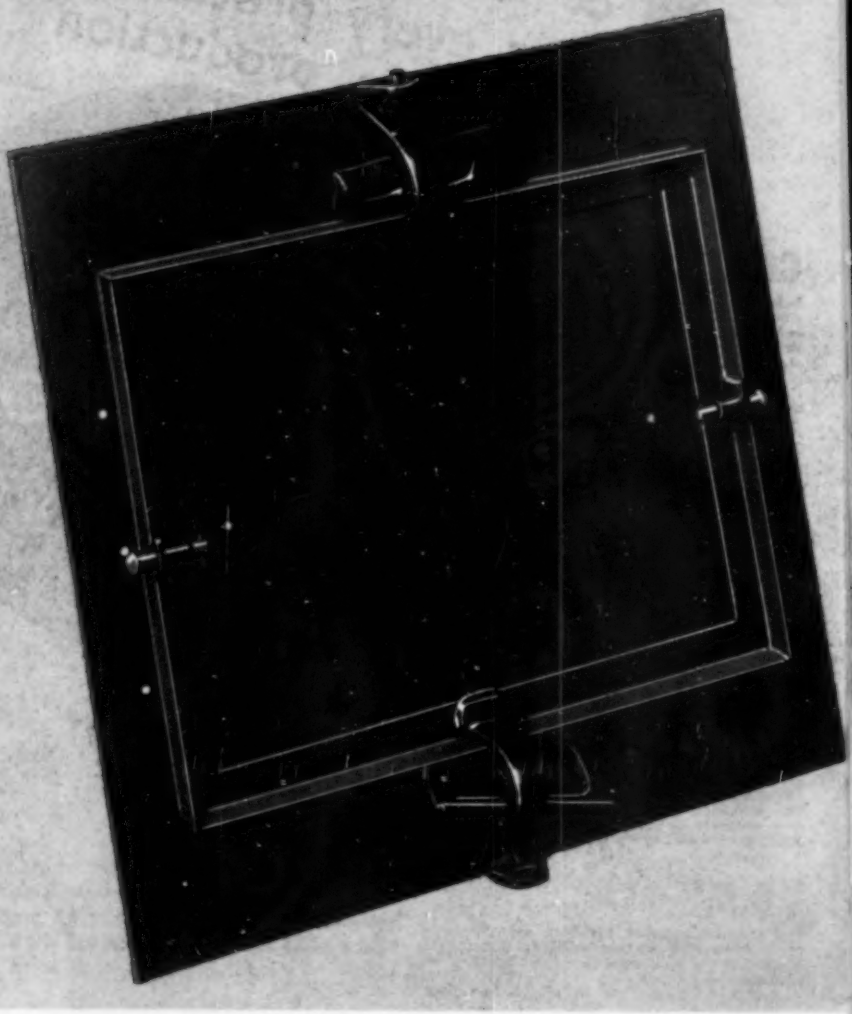

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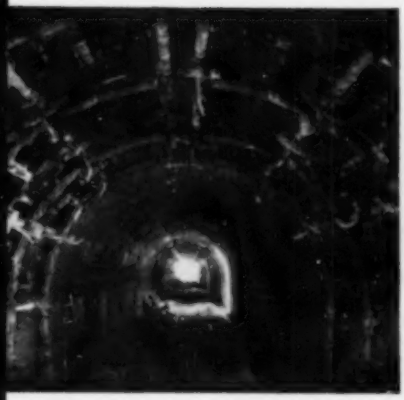

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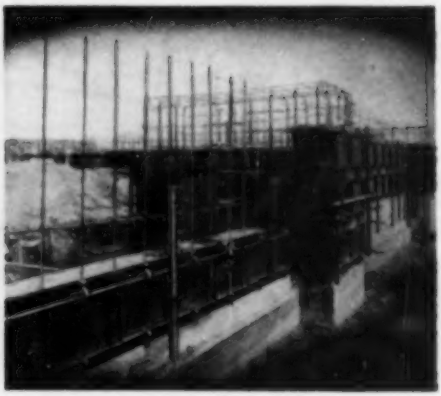

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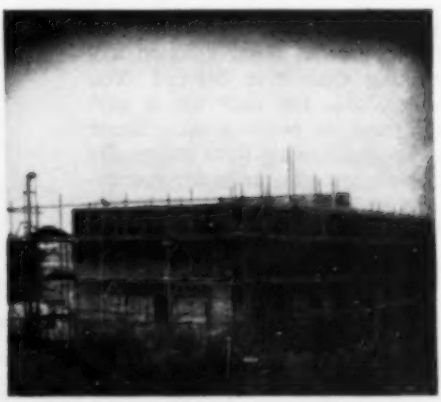

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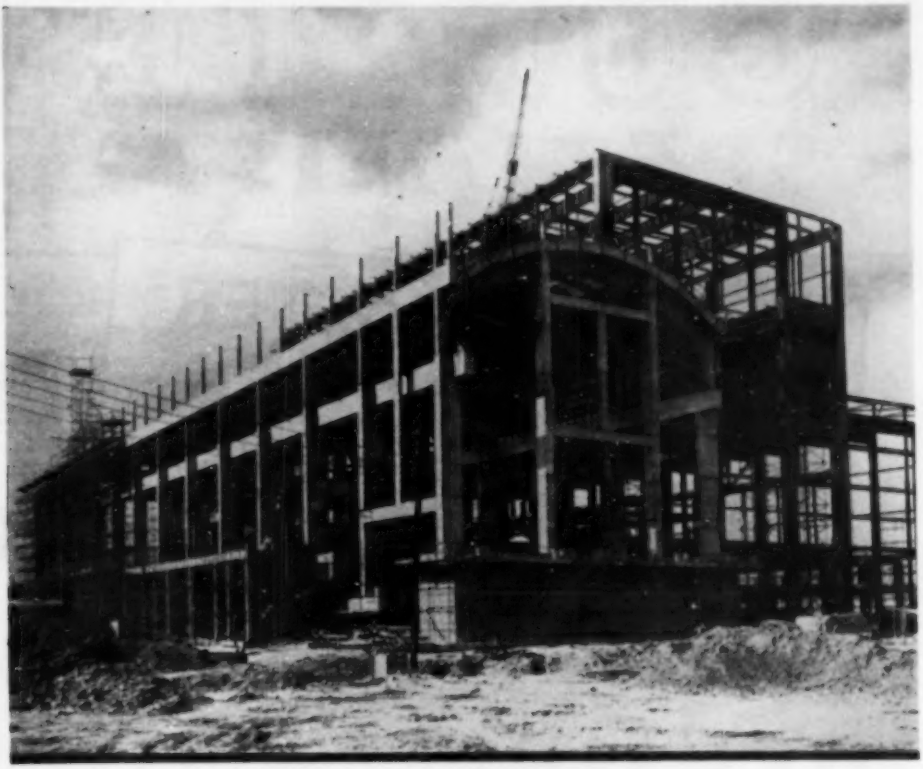

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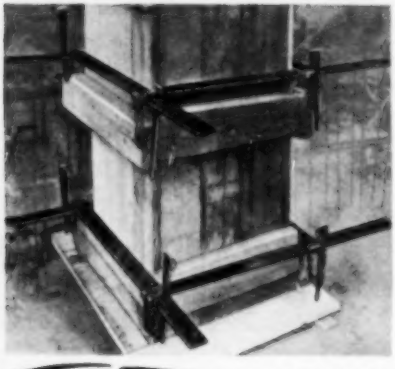

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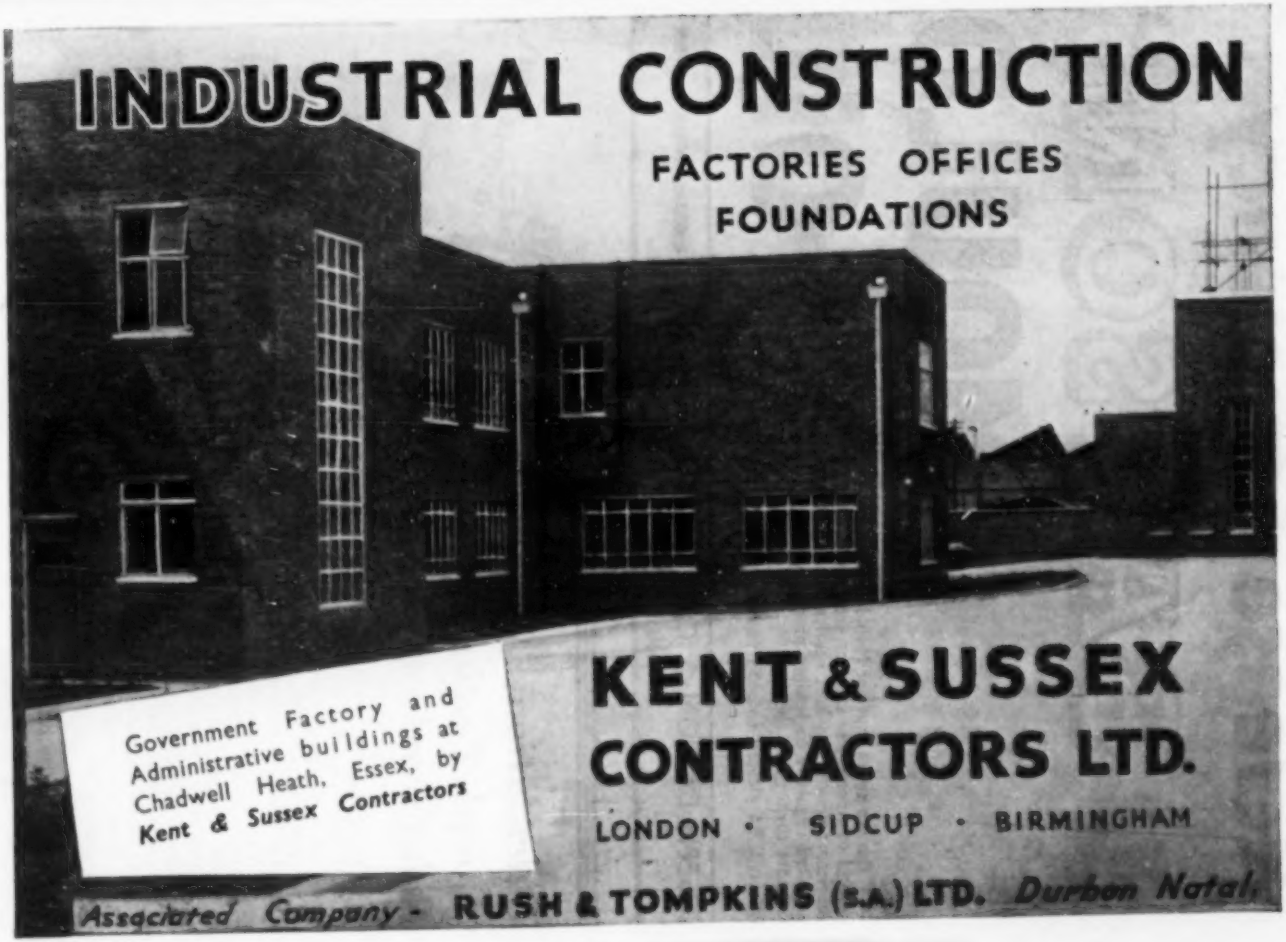
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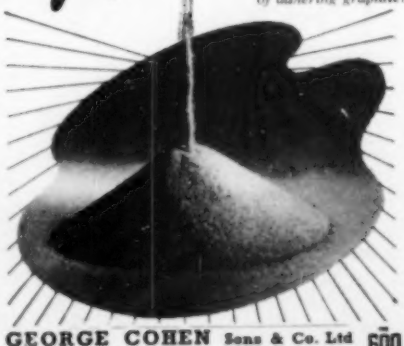

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CONCRETE
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Volume XLV. No. 4. LONDON, APRIL, 1950

EDITORIAL NOTES

A Code of Practice for Concrete Slabs and Stairs.

ALTHOUGH a slab is generally the most simple reinforced concrete member to design, many of the problems cannot be solved by statical methods and are therefore commonly dealt with by what may be called judicious guesses. The results of tests are sometimes available to assist the designer in his judgment, but experiment cannot deal with all the variations of every-day problems such as the effect of concentrated loads on slabs spanning in one direction, limitation of the ratio of the depth to the span of slabs, the proportions of the parts of a ribbed floor, and the amount of transverse reinforcement, if any, required in a solid slab. Rules for the solution of these and many other matters are given in a new British Standard code of practice, some of the requirements of which are given elsewhere in this number. The code deals with the design and construction of suspended reinforced concrete floors and roofs constructed of cast-in-situ solid slabs spanning in one or two directions, flat slabs, slabs comprising ribs with or without intervening blocks, precast slabs, and filler-joist slabs. Stairs, the design of which usually entails the application of rule-of-thumb methods, are also included.

Many of the recommendations, such as those relating to concrete materials, reinforcement, working stresses, slabs spanning in two directions, and flat slabs are repetitions of the corresponding parts of the British Standard Code for the "Structural Use of Normal Reinforced Concrete in Buildings." One of the few instances of the new and the previous recommendations being at variance is in the amount of reinforcement (or distributing bars) to be provided at right-angles to the main reinforcement in a solid slab spanning in one direction. This matter generally seems to present difficulties to compilers of codes and regulations. The regulations of the London County Council issued in the year 1915 required the cross-sectional area of distributing bars to be 0·08 per cent. of the effective cross-sectional area of the concrete. The D.S.I.R. code of 1934 recommended that the area of the distributing bars should be one-fifth of the area of the main reinforcement. The London County Council By-laws of 1938 require distributing bars to be one-tenth of the area of the main reinforcement, which is the amount recommended in the code under review, while the British Standard Code of 1948 for ordinary reinforced concrete recommends that distributing bars be 0·1 per cent. of the gross cross-sectional area of the concrete.

The empirical rules for the determination of the width of slab which can be considered to carry a concentrated load take into account the width of the loaded

area and the position of the load in respect to the longitudinal and transverse edges of the slab. The omission of the thickness of the slab may not matter if the load is spread over a large area of a thin slab, but in other cases the results may be too conservative. The restriction of the rules to simply-supported slabs and the omission of rules for shearing forces detract from what might otherwise be very useful guidance. The fact that the length (as opposed to the width) of the loaded area is not taken into account is a more serious omission that leads to different results being obtained in the limiting case of the load being concentrated on a strip extending over the entire span of the slab, depending on whether the load is considered as a single load or as a combination of two or more loads.

The empirical rules for the distribution of load on, and the effective spans of, stairs supported in various ways are logical for ordinary cases, although the results are conservative, and should be applied with caution in extreme cases such as stairs with few risers in a rectangular well.

The recommendation that ribbed floors be designed as freely supported is also a little severe in view of the fact that so much reinforcement (25 per cent. of the main reinforcement) is to be provided to resist the negative bending moment over the support. A reduction of, say, 12½ per cent of the free positive bending moment would not be unreasonable in these circumstances. In the construction of ribbed floors with hollow blocks between the ribs, it is quite rightly stated that the concrete placed in situ should not be consolidated by mechanical vibration, but the requirement that the joints between the blocks in " alternate rows " (presumably meaning " adjacent rows ") should be staggered is an unnecessary refinement and not in accordance with common practice. The recommended minimum widths of bearing of precast floor members on concrete supports (3 in.) and on brickwork (4 in.) seem to be too small to allow for the exigencies of construction, especially as even these widths may be reduced in some circumstances.

In spite of the foregoing criticisms, the code will be of service to designers who have not formulated sound rules of their own for some of the problems dealt with. As some of the recommended rules are those already used by many engineers, the document justifies the title of a code of practice. It could be much improved, as could many other codes, if in place of many recommendations to provide " adequate," " sufficient," " enough," and the like strengths or other properties, more precise guidance were given to establish what is adequate, etc., even if such guidance took the form of explanatory notes and examples not necessarily forming a part of the code but in, say, an appendix. An example of loose wording is that the topping of ribbed floors should be designed to carry loads, such as partitions placed between the ribs, from rib to rib, and that " particular attention should be paid to all stresses in the topping." Does this mean that the designer must estimate, in addition to the transverse bending stresses due to the partition, the transverse stress due to the longitudinal bending, and shrinkage and other secondary stresses in a thin topping the actual thickness of which may vary considerably ? The lengthy repetition of parts of the previous code for reinforced concrete, to which this code is supplementary, will be avoided no doubt when the two codes are combined in a single document. It was suggested in these notes recently that one code embodying the requirements of the several codes relating to walls would assist easy reference, and those parts relating to reinforced concrete walls could with advantage be included in the combined code.

The New Electricity Station at West Ham.

THE erection of the new electricity generating station "B" at West Ham was commenced by the West Ham Corporation but is now under the British Electricity Authority. The power-house is being constructed in three stages, each to contain two 30,000-kw. turbo-generators. The first stage is almost completed and is expected to be in operation early this year. The construction of the second stage has commenced. The principal concrete work in the power-house is in the foundation raft and the piers supporting the turbo-generators; auxiliary structures of reinforced concrete are a building for offices and electrical equipment, coal-mixing bunkers, ash bunkers, culverts, three cooling towers (two of which are completed), a chimney, and a coal wharf.

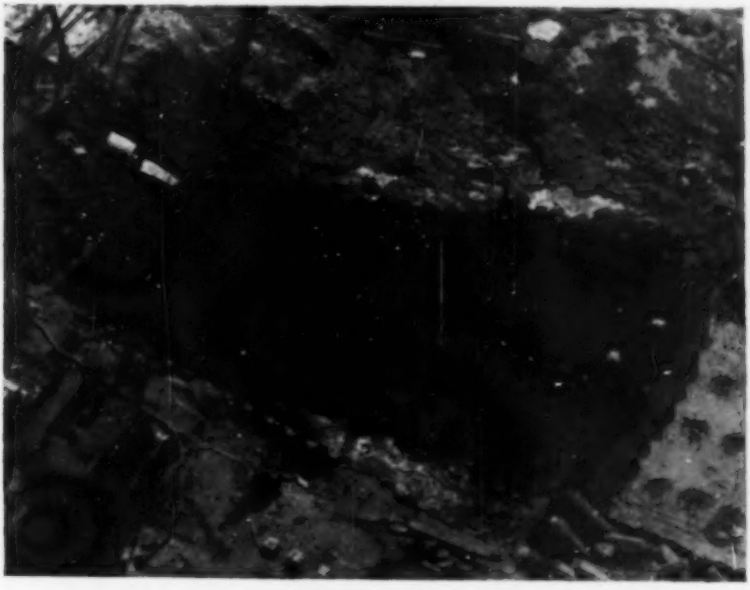


Fig. 1.—Small Cofferdam with External Ties.

Foundations and Cofferdams.

The area of the site, which is bounded on the south by the river Lea, is about 20 acres. The level of the ground water is about 12 ft. below the surface, and a stratum of ballast 6 ft. to 10 ft. thick occurs about 15 ft. below the surface. London clay lies below the ballast. Most of the foundations, which are constructed within cofferdams, are on the ballast; the pressure under the power-station raft is about 3 tons per square foot and elsewhere from 2 tons to 3 tons per square foot. Steel sheet-pile cofferdams are used, the piles being driven into the clay and, where possible, extracted upon completion of the construction within them. The cofferdams were kept dry by continuous pumping. After three weeks of pumping, a 6-in. pump was sufficient to deal with the inflow into the large cofferdam for the power-house foundation which is 300 ft. long and 120 ft. wide. Generally the cofferdams were stiffened by internal timbering, but for the cofferdam (*Fig.* 1) for the excavation, which is 30 ft. deep, for the main sump and pump chamber, the tops of the steel sheet-piles were tied back externally by steel ties to short steel sheet-piles driven for this purpose. The extra cost of this

method is said to be repaid by the ease with which shuttering and concreting proceed in an excavation almost unhampered by timbering. The absence of timbering also enabled most of the excavation to be performed by a drag-line excavator.

The front wall of the reconstructed coal wharf, which is described later, was built within a cofferdam driven in two successive parts, the excavation within the cofferdam extending 25 ft. to 30 ft. below high-tide level. The sheet-piles for part of this cofferdam, which was in the river works on the banks of the river on this site, and old foundations and disused sewers, 4 ft. and more in diameter and 20 ft. below the ground, had to be removed before the piles could be driven.

About 1200 precast piles have been driven, most of them by a hammer operating on an ordinary frame, but where it was impracticable for the frame to approach closely enough to the position of the pile, some of the precast piles were driven from a drag-line excavator adapted for this purpose (*Fig.* 2). Steel leaders, long enough to take a 40-ft. pile, were

Fig. 2.—Excavator Converted to Pile Driver.

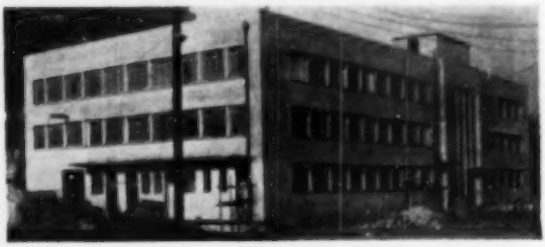


Fig. 3.—Office and Electrical Control Building.

Lea, were driven into the heads of old timber piles in the bed of the river. To extract the steel piles it was necessary to exert an upward pull of nearly 200 tons, which was done by two 100-tons jacks. After lifting the piles about 2 ft., extraction by ordinary hammers was possible.

The position of live cables from the old power station made it impossible to drive continuous steel sheet-piles for a cofferdam within which deep foundations for the transformer house could be constructed, and therefore precast reinforced concrete piles are used for this foundation. Similar piles 14 in. square and 40 ft. long are provided to carry the deck of the coal wharf. There was at one time a sewerage suspended from the 60-ft. jib of the excavator. The leaders were steadied at the bottom by attachment to a triangular steel frame projecting from the bed-plate of the excavator. About 100 piles were driven by this equipment. A 3-tons drop-hammer was used, and the piles generally were driven to a set of five blows to an inch when the drop was 4 ft.

Shuttering.

Timber shuttering is generally used with pressed-wood and plywood lining for external faces. Steel-plate shuttering is used for the soffits of floor slabs and other horizontal slabs. Some of the timber shuttering is made on the site, although most of it is made elsewhere to designs prepared in a drawing office on the site. A feature of the drawings is a number of views showing details of the shuttering to be erected and the extent of concrete to be placed in successive stages of the construction of each structure. About 750,000 sq. ft. of concrete face have been shuttered to date.

The main office block and electrical control building (*Fig.* 3) is a reinforced concrete structure 180 ft. long, 70 ft. wide, and 40 ft. high, of cast-in-situ construc-

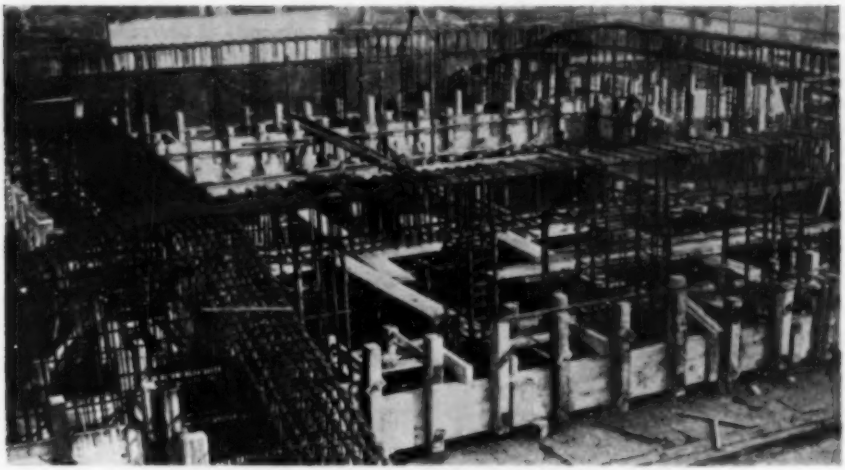


Fig. 4.—Construction of Roof of Control Room.

tion throughout including the heads and sills of the windows and the mullions. The central feature is a group of reinforced concrete columns of trapezoidal cross-section, the spaces between the columns being filled with glass bricks. Plywood in sheets 8 ft. by 4 ft. and $\frac{3}{4}$ in. thick was used to line the shuttering for the walls, care being taken to align the panels correctly as slight variations of line

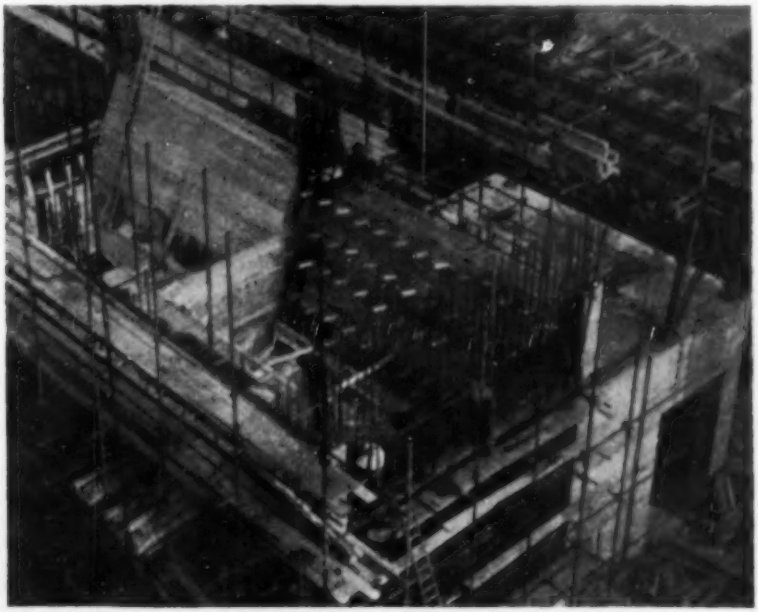


Fig. 5.—Shuttering for Ash Bunkers.

would be apparent on the large plain face of the walls. After the shuttering had been stripped, the outer face of the concrete was cleaned and rubbed down by hand with carborundum stones. The main beams of the roof of the control room span 52 ft. Tubular steel scaffolds erected on the floor were provided to support the boxes for the roof beams, the shuttering for the slabs being suspended between adjacent scaffolds. The reinforcement of the main beams and other roof beams and columns, and shuttering for parts of this work, are shown in *Fig.* 4.

The ash bunkers comprise three compartments each 24 ft. by 20 ft. and with pyramoidal hoppers. The height of the structure is about 100 ft., and the total weight of about 2000 tons is carried on eight columns supported on precast concrete piles. Each of the most heavily loaded columns supports a load (including that due to wind) of about 600 tons and a bending moment of about 285,000 ft.-lb. The depth of each compartment is 38 ft. 3 in. and the contents (wet ashes) are assumed to be a liquid weighing 65 lb. per cubic foot. The longitudinal walls are 15 in. thick and the transverse walls 18 in. There are more than 200 tons of reinforcement in the structure. The hopper bottoms slope at angles of 59 deg. and 67 deg., and the correct distance between the shuttering for the upper and lower faces of the sloping slabs was maintained by small precast concrete struts (*Fig.* 5). The upper faces of the sloping slabs are indented to facilitate the flow of the ashes, and the trapezium-shape panels of shuttering for these faces (*Fig.* 6) comprised a wooden frame to which plywood was attached to form the indents.

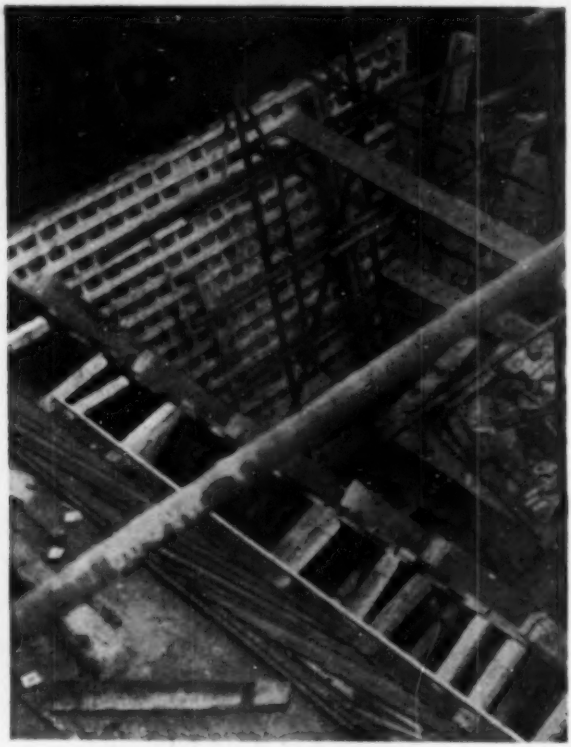


Fig. 6.—Shuttering for Indented Hopper Slopes of Ash Bunker.

Concrete.

An automatic batching and mixing plant is used (*Fig.* 7). The steel bins contain 60 tons of aggregates, which are measured by weight. The cement is delivered in bulk by motor lorries and is discharged into a cylindrical steel container of a capacity of 70 tons. The cement is transported to one compartment of the steel bins by a screw conveyor and bucket elevator. The concrete mixer, the capacity of which is 1 cu. yd. of mixed concrete, has a tilting drum and can produce about 30 cu. yd. of concrete in an hour. The mixer discharges into a hopper from which concrete pumps are fed, or which discharges into lorries or other containers for use in parts of the work where small quantities are required. There will be about 75,000 cu. yd. of concrete in the works, most of which is distributed by concrete pumps, the pipe-lines for which extend up to 1000 ft. The concrete in the upper part of the ash bunkers is lifted to a height of about 100 ft. above the ground by a 4-in. pump.

Construction of the foundations in the first stage of the power-house started in May, 1947, and was completed in eight months. In each stage of the work, which is 300 ft. long and 120 ft. wide, there are about 17,000 cu. yd. of concrete, the average quantity placed weekly being about 500 cu. yd., which is also the average rate for the works as a whole. In the large plain concrete foundations the greatest quantity placed was from 1000 cu. yd. to 1200 cu. yd. a week for a period of two months. A part of a reinforced concrete foundation raft is shown in course of construction within a cofferdam in *Fig.* 8.

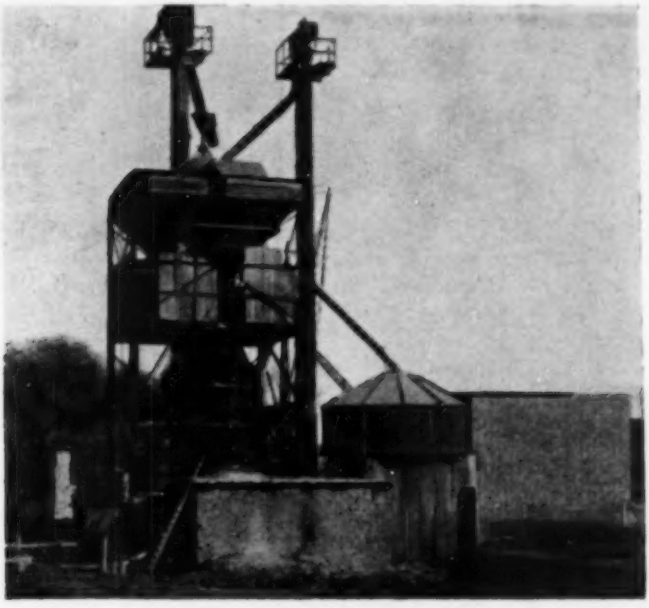


Fig. 7.—Batching and Mixing Plant.

The two piers supporting the turbo-generators in each stage of the power house are about 26 ft. high, and each contains 150 cu. yd. of concrete, which was placed in one operation. The first pier was concreted in 17½ hours, using a 6-in. concrete pump. The second pier was concreted in 10½ hours using a 6-in. pump and a 4-in. pump; about 20 men were employed placing and vibrating.

Except for small quantities of concrete that are tamped by hand, all concrete is consolidated by immersion electrical vibrators. In the superstructures the

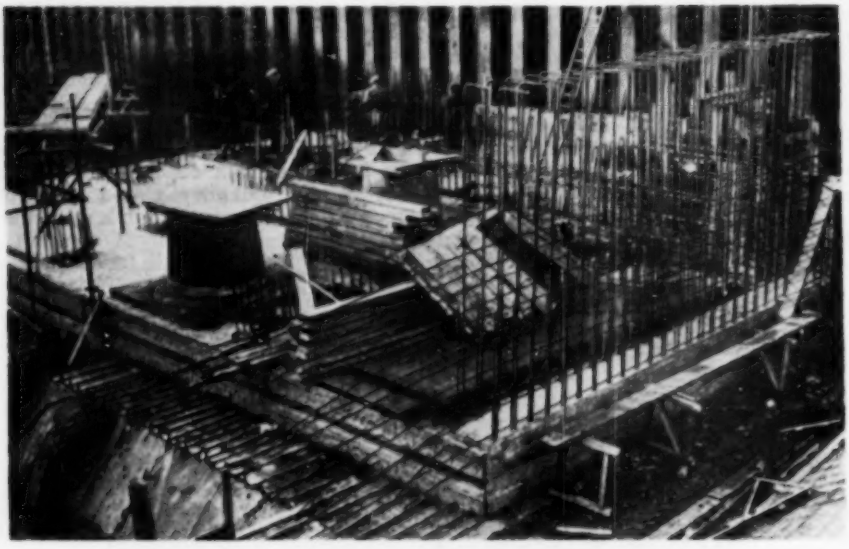


Fig. 8.—Construction of Foundation within Cofferdam.

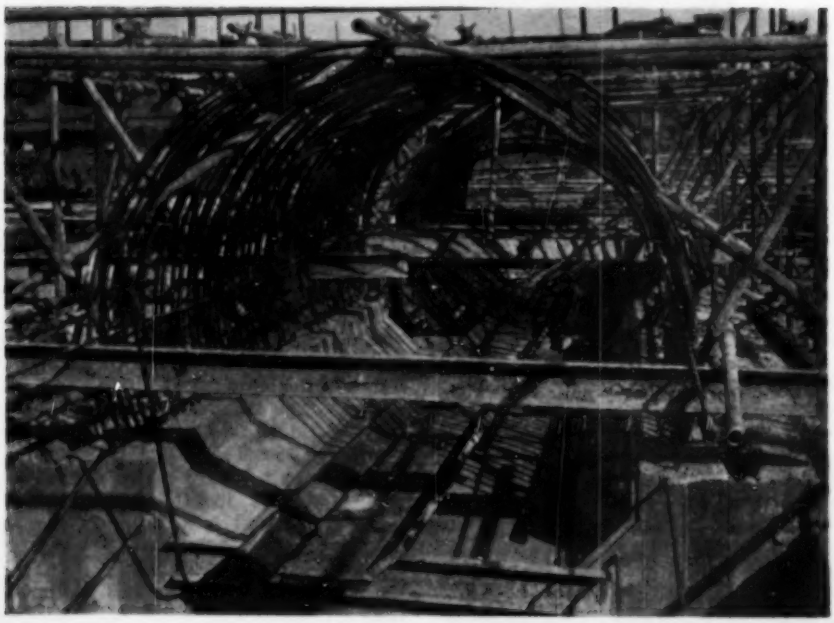


Fig. 9.—Culvert in Course of Construction.

cement content of the concrete is 520 lb. in a cubic yard of mixed concrete.

Culverts.

When the station is working to full capacity, about 8,000,000 gallons of cooling water will be required hourly. The water will be circulated through reinforced concrete culverts, the total length of which is about 3000 ft. Most of the culverts are about 6 ft. square. About 1000 ft. of culvert is being constructed as a double culvert with a gap 12 in. wide between the inlet (cold water) culvert and the outlet (warm water) culvert to reduce the transference of heat. At one point, the double culvert makes a right-angle bend where it passes below a

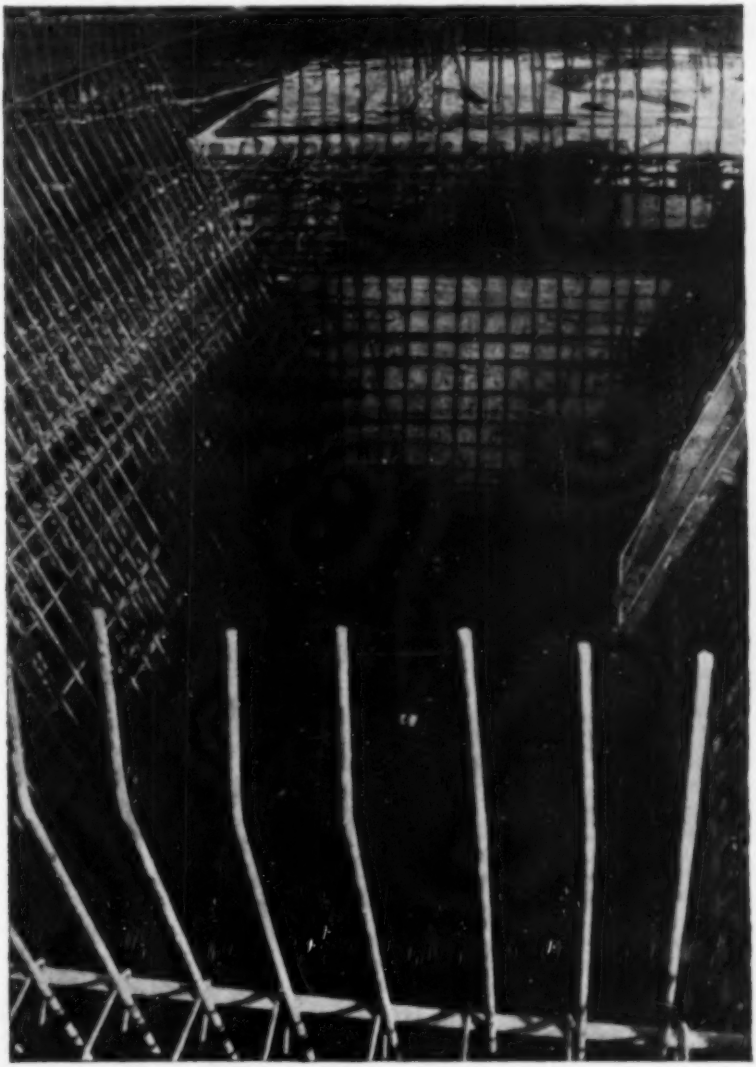


Fig. 10.—Reinforcement of Tippler Pit.

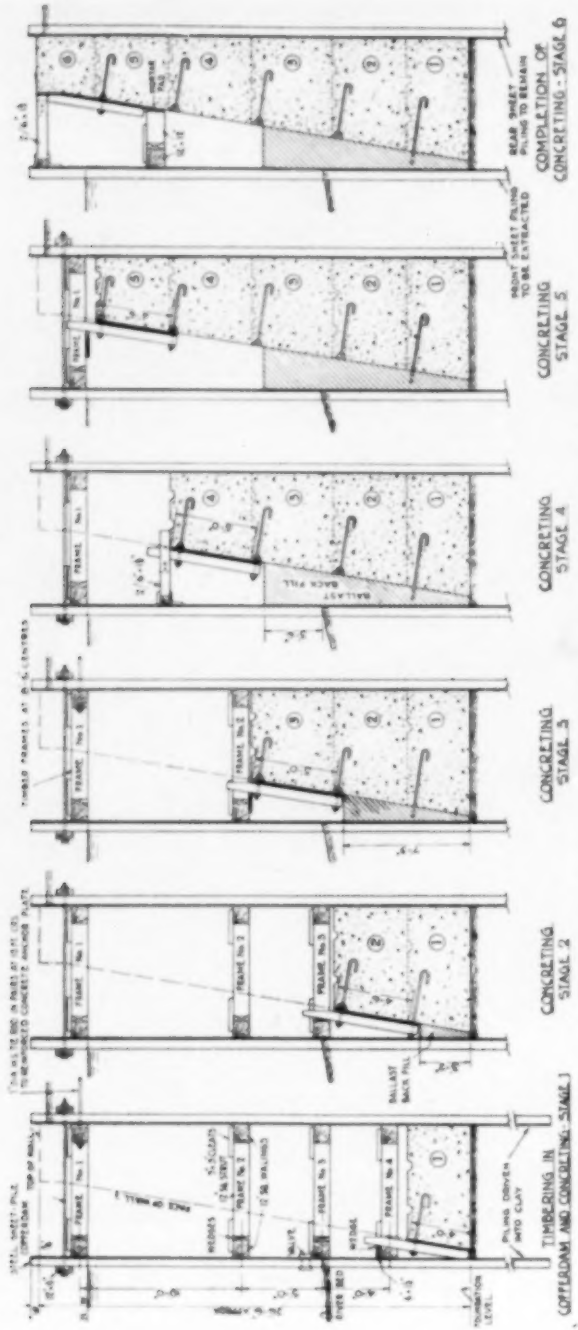


Fig. 11.—Construction of the Wall of the Coal Wharf.

group of live cables from the old power station. As the space between the top of the concrete culvert and cables is only 4 in., the cables were slung from a temporary steel bridge while the culverts were constructed. *Fig.* 9 shows a single reinforced concrete culvert in course of construction. The invert has been concreted and the reinforcement for the roof slab is in position. The shuttering for the splay at the bottom of the walls, and the method of supporting the reinforcement from steel scaffold tubes, are seen.

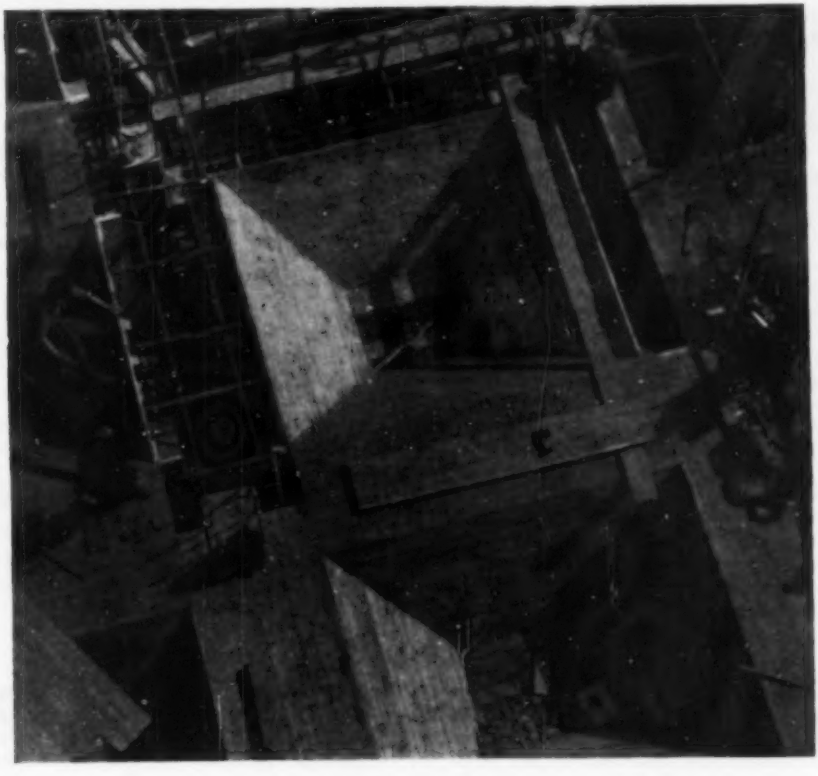


Fig. 12.—Shuttering for Hoppers of Coal Mixing Bunkers.

Coal Wharf.

The completed station will consume about 2000 tons of coal daily. Rail-borne coal will be supplied to conveyors from a tippler-pit, the reinforcement of one hopper of which is seen in *Fig.* 10. Coal delivered by barge is unloaded at a new wharf, which replaces and extends into part of the old wharf and is about 300 ft. long. Near the wharf are the coal-mixing bunkers, the shuttering for the hoppers of which is shown in *Fig.* 12.

The new wharf comprises a front wall of plain concrete founded on ballast, and a reinforced concrete deck, 63 ft. wide, carrying a crane and supported on groups of 14-in. square precast piles 40 ft. long driven through the ballast into the underlying clay. A continuous row of 20-in. by 10-in. precast reinforced concrete sheet-piles is driven at the back of the wharf. The sheet-piles do not interlock but are connected by vertical circular grout keys. A layer of chalk is deposited on the bed of the river in front of the wharf to form a barge-berth. Timber

rubbing pieces are attached to the face of the wall. A part of the old wharf has also been reconstructed, the old brick wall being retained and a new reinforced concrete deck supported on piles having been built.

The method of constructing the new plain concrete wall within steel sheet-pile cofferdams is shown in *Fig.* 11. The cofferdam was strutted with four horizontal timber frames which were extracted one at a time as the six successive lifts of concrete were placed. Before each of the lower frames was removed, the stability of the cofferdam was maintained by placing ballast between the front row of steel sheet-piles and the face of the hardened concrete. Before the top frame was removed the sheet-piles were strutted off the part of the wall already built, and, to ensure a firm bearing, a pad of mortar was placed between the end of the wooden strut and the sloping face of the concrete wall. The cofferdams were anchored by ties attached to the top of the sheet-piling at 15-ft. centres, each tie comprising a pair of 1-in. steel rods connected to a 6-in. vertical reinforced concrete anchor plate, 4 ft. 6 in. long and 3 ft. high, buried in the ground about 32 ft. behind the cofferdam.

The concrete was placed against the rear line of steel sheet-piles, but the front face of the wall was shuttered. The same panels of shuttering were used for the sloping front face of the wall for each lift. Four 1¼-in. boards 12-in. wide were screwed to 6-in. by 1½-in. cleats attached to double studs of 6-in. by 3-in. timber placed at 1 ft. 9 in. centres. Other 6-in. boards were added after the first and second lifts. Two hook-bolts were passed through each stud and were eventually embedded in the concrete. A hardwood collar on each bolt formed a recess in the face of the concrete. After removal of the shuttering, the bolt was cut off at the bottom of the recess, which was then filled with mortar. This method prevented rust stains on the face of the wall. The means by which the tops of the studs were kept in position is shown in *Fig.* 11.

When the wall was completed the steel sheet-piles forming the front wall of the cofferdam were extracted, but those forming the back wall were left in position. Filling was deposited behind the wall, and was used in place of shuttering for the soffit of the reinforced concrete deck. Trenches were formed in the filling for the beams and pile-caps, the sides of the trenches being lined with boards.

The consulting engineers for the civil engineering work are Sir Alexander Gibb & Partners. The main contractors are Taylor-Woodrow Construction, Ltd. The reinforced concrete chimney, which is 325 ft. high and is carried on the steelwork of the roof of the boiler house, was erected by Messrs. Tileman & Co., Ltd. The construction of the first of the reinforced concrete cooling towers, the contractors for which were Messrs. J. L. Kier & Co., Ltd., was described in this journal for December, 1948.

The Thermal Expansion of Concrete.

TESTS to determine the coefficient of thermal expansion of aggregates and of concretes made with various aggregates and cements are described by Mr. D. G. R. Bonnell and Mr. F. C. Harper in the Journal of the Institution of Civil Engineers for February, 1950. A complete report is to be published later by the Building Research Station.

The temperatures in the tests were from 32 deg. F. to 104 deg. F. The coefficients of aggregates are from $2{\cdot}1 \times 10^{-6}$ to $6{\cdot}5 \times 10^{-6}$ per deg. F., siliceous materials having the highest coefficients, limestones the lowest, and igneous materials being intermediate. The coefficients for concrete are from $3{\cdot}4 \times 10^{-6}$ to $7{\cdot}3 \times 10^{-6}$ per deg. F., the magnitude depending largely on the aggregate. Concretes with siliceous aggregate have the highest coefficients, those with limestone aggregate the lowest, and those with igneous aggregate are intermediate.

The coefficient of thermal expansion of neat hydrated cement is greater than that of any of the aggregates tested. Concretes rich in cement tend to have greater coefficients than lean concrete, but the difference is not great. The age of the concrete, the method of curing, and the water content of the mixture have only small effects on the thermal expansion. Very dry and very wet concretes have similar coefficients of expansion, but the coefficients are smaller than those of partly-dry concrete.

Ribbed Slabs Spanning in Two Directions.

By J. C. HELAL, B.Sc.(Eng.), A.M.I.Struct.E. (CAIRO.)

PANELS of hollow-tile slab and similar types of ribbed floors and flat roofs, if supported on four sides, can be designed to span in two directions at right-angles by providing ribs in both directions. As the ordinary Grashof and Rankine coefficients may be too severe for this type of construction, and also as it may not conform sufficiently to the definition of a thin plate for the more theoretical analyses to apply, the load on such ribbed slabs can be distributed according to the Grashof and Rankine method and the ribs then designed as a system of interlacing beams in accordance with the simplified method derived by Professor Saliger. The method is applicable particularly to hollow-tile slabs in which the thickness of the cast-in-situ top slab is 20 per cent. to 30 per cent. of the total thickness of the slab, and in such cases gives a design that is about as economical as one based on coefficients derived from the theory of elasticity.

In Professor Saliger's method the maximum bending moments on beams spanning in one direction depend on the ratios of the greatest deflection of the

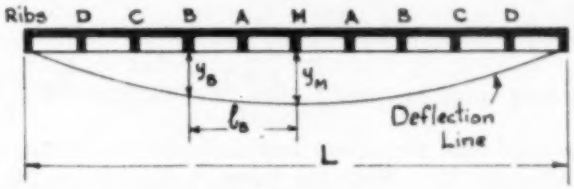


Fig. 1.

beams. If in a series of ribs (*Fig.* 1) the maximum bending moment on the middle rib M is M_M, the maximum bending moment M_N in any rib N is $K_N M_M$. The coefficient K_N depends on the number of ribs n and on the distance l_N of the rib N from the middle rib M, and is given by

$$K_N = \frac{y_N}{y_M} = \frac{16\alpha_N}{5}(1 - 2\alpha_N^2 + \alpha_N^3) \quad . \quad . \quad . \quad (1)$$

where $\alpha_N = \frac{l_N}{L}$. The average value K_{AV} of the values of K_N for each rib in a slab is $\frac{\Sigma_1^n K_N}{n}$. The average bending moment M_{AV} in the ribs is $K_{AV} M_M$. If $R_N = \frac{K_N}{K_{AV}}$, the bending moment M_N in any rib N is $R_N M_{AV}$.

The maximum bending moment M_M is calculated in the ordinary way using the Grashof and Rankine coefficients k_S and k_L applicable to the ratio of the short span to the long or, for a slab in which the ribs are spaced equally in both directions, to the ratio of the numbers of bays in the short and long spans. The number of bays is $n + 1$ if n is the number of ribs excluding the end ribs.

For a panel of slab freely supported along four edges the bending moments are:

For the short span, L_S: $M_M = \frac{k_S W L_S^2}{8}$; $M_{AV} = \frac{K_{AV} k_S W L_S^2}{8}$;

TABLE I.—HOLLOW-TILE SLABS SPANNING IN TWO DIRECTIONS.

VALUES OF COEFFICIENTS R								VALUES OF COEFFICIENTS C											
NO. OF BAYS	R_M	R_A	R_B	R_C	R_D	R_E	R_F	NO. OF BAYS IN LONG SPAN	NO. OF BAYS IN SHORT SPAN										
									4	5	6	7	8	9	10	11	12	15	14
14	1·46	1·42	1·32	1·14	0·92	0·64	0·35	14							0·543 0·146	0·493 0·195	0·455 0·250	0·397 0·290	0·363 0·363
15	–	1·43	1·36	1·20	0·97	0·68	0·36	15						0·562 0·132	0·511 0·183	0·476 0·236	0·401 0·292	0·346 0·346	
12	1·44	1·39	1·25	1·02	0·73	0·39		12					0·580 0·119	0·529 0·164	0·467 0·225	0·403 0·285	0·347 0·347		
11	–	1·41	1·31	1·09	0·78	0·41		11					0·547 0·155	0·476 0·225	0·416 0·286	0·349 0·349			
10	1·42	1·55	1·15	0·85	0·46			10				0·570 0·142	0·500 0·210	0·427 0·281	0·353 0·353				
9	–	1·38	1·22	0·92	0·49			9			0·592 0·124	0·519 0·198	0·437 0·276	0·356 0·356					
8	1·33	1·28	0·99	0·54				8			0·548 0·180	0·456 0·271	0·361 0·361						
7	–	1·35	1·07	0·60				7		0·582 0·160	0·470 0·270	0·367 0·367							
6	1·35	1·16	0·68					6	0·628 0·133	0·507 0·251	0·375 0·375								
5	–	1·25	0·77					5	0·549 0·234	0·387 0·387									
4	1·24	0·88						4	0·404 0·404										

IN EACH PAIR OF COEFFICIENTS:
UPPER FIGURE IS $C_S = k_S K_{AV}$ (SHORT SPAN)
LOWER FIGURE IS $C_L = k_L K_{AV}$ (LONG SPAN)

For the long span, L_L: $M_M = \frac{k_L W L_L^2}{8}$; $M_{AV} = \frac{K_{AV} k_L W L_L^2}{8}$;

where W is the total load per square foot multiplied by the distance between the centres of adjacent ribs. Since the values of K_{AV}, k_S, and k_L depend only on the number of ribs in each direction, it is convenient to substitute $C_S = K_{AV} k_S$ and $C_L = K_{AV} k_L$. Therefore

M_{AV} (short span) $= 0{\cdot}125 C_S W L_S^2$; M_{AV} (long span) $= 0{\cdot}125 C_L W L_L^2$.

The coefficients required for the design of a slab are therefore R and C only, and values of these are given in *Table* No. 1 for slabs having the same or different numbers of bays in each direction.

Calculation of Coefficients.

As an example of the method of calculating the coefficients R and C, consider a panel having eight bays in one direction and ten in the other. Denoting the nine ribs spanning across the short way by M, A, B, C, and D, as in *Fig.* 2, $\alpha_A = 0{\cdot}1$, $\alpha_B = 0{\cdot}2$, $\alpha_C = 0{\cdot}3$, and $\alpha_D = 0{\cdot}4$. By successive substitution in (1), $K_M = 1{\cdot}0$, $K_A = 0{\cdot}950$, $K_B = 0{\cdot}812$, $K_C = 0{\cdot}595$, and $K_D = 0{\cdot}320$. Therefore

$$K_{AV} = \frac{1}{9}[2(0{\cdot}950 + 0{\cdot}812 + 0{\cdot}595 + 0{\cdot}320) + 1{\cdot}0] = 0{\cdot}706$$

$$R_M = \frac{K_M}{K_{AV}} = \frac{1{\cdot}0}{0{\cdot}706} = 1{\cdot}42\,; \quad R_A = \frac{0{\cdot}950}{0{\cdot}706} = 1{\cdot}35\,; \quad R_B = 1{\cdot}15\,; \text{ etc., as in}$$

Table No. 1. Similarly, for the seven ribs spanning across the long way, $K_M = 1{\cdot}0$, $K_A = 0{\cdot}927$, $K_B = 0{\cdot}711$, and $K_C = 0{\cdot}389$, from which $K_{AV} = 0{\cdot}722$, $R_M = 1{\cdot}38$, $R_A = 1{\cdot}28$, etc., as in *Table* No. 1.

The coefficients k_S and k_L are obtained from the well-known formula of

Grashof and Rankine, and for a slab having a ratio of length to breadth of $\frac{1.9}{8}$ (1·25), k_S is 0·71 and k_L is 0·29. Therefore $C_S = 0{\cdot}706 \times 0{\cdot}71 = 0{\cdot}5$, and $C_L = 0{\cdot}722 \times 0{\cdot}29 = 0{\cdot}21$, as in *Table* No. 1.

Example.

Design a slab 20 ft. by 16 ft. (*Fig.* 2) with ribs spaced at 2-ft. centres in both directions, the total load including the weight of the slab being 140 lb. per square foot. Therefore $L_S = 16$ ft., $L_L = 20$ ft., and $W = 2 \times 140 = 280$ lb. The number of bays are ten and eight, for which the coefficients in *Table* No. 1 are $C_S = 0{\cdot}5$ and $C_L = 0{\cdot}21$.

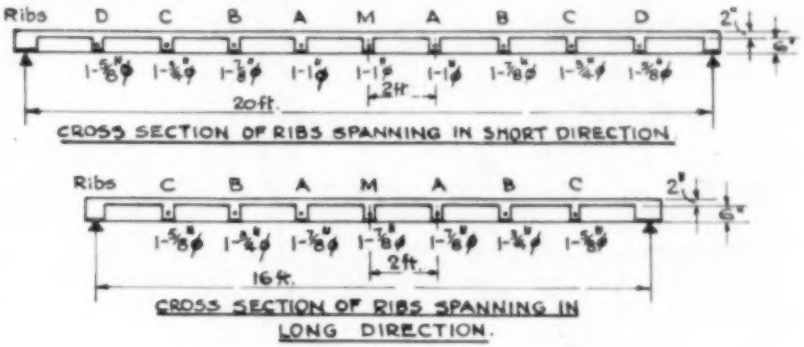

Ribs D C B A M A B C D
2"
6"
1-5/8"ϕ 1-3/4"ϕ 1-7/8"ϕ 1-1"ϕ 1-1"ϕ 1-1"ϕ 1-7/8"ϕ 1-3/4"ϕ 1-3/8"ϕ
2ft.
20ft.
CROSS SECTION OF RIBS SPANNING IN SHORT DIRECTION.
Ribs C B A M A B C
2"
6"
1-5/8"ϕ 1-3/4"ϕ 1-7/8"ϕ 1-7/8"ϕ 1-7/8"ϕ 1-3/4"ϕ 1-5/8"ϕ
2ft.
16ft.
CROSS SECTION OF RIBS SPANNING IN LONG DIRECTION.

Fig. 2.

For ribs spanning in the short direction:

$M_{AV} = 0{\cdot}5 \times 280 \times 0{\cdot}125 \times 16^2 \times 12 = 54{,}000$ in.-lb. per rib.

Average area of reinforcement (assuming a lever arm of 5·5 in.)

$$= \frac{54{,}000}{5{\cdot}5 \times 18{,}000} = 0{\cdot}55 \text{ sq. in. per rib.}$$

With the values of R in *Table* No. 1:

Reinforcement in rib M: 1·42 × 0·55 = 0·78 sq. in.; one 1-in. bar.
" " " A: 1·35 × 0·55 = 0·74 " " " 1-in. "
" " " B: 1·15 × 0·55 = 0·63 " " " $\frac{7}{8}$-in. "
" " " C: 0·85 × 0·55 = 0·47 " " " $\frac{3}{4}$-in. "
" " " D: 0·46 × 0·55 = 0·25 " " " $\frac{5}{8}$-in. "

For ribs spanning in the long direction:

$M_{AV} = 0{\cdot}21 \times 280 \times 0{\cdot}125 \times 20^2 \times 12 = 35{,}400$ in.-lb. per rib.

Average area of reinforcement (lever arm = 5 in.) = 0·39 sq. in. per rib.

Reinforcement in rib M: 1·38 × 0·39 = 0·54 sq. in.; one $\frac{7}{8}$-in. bar.
" " " A: 1·28 × 0·39 = 0·50 " " " $\frac{7}{8}$-in. "
" " " B: 0·99 × 0·39 = 0·39 " " " $\frac{3}{4}$-in. "
" " " C: 0·54 × 0·39 = 0·21 " " " $\frac{5}{8}$-in. "

The foregoing calculations assume that the total thickness of the slab is 8 in., the compression flange being 2 in. thick. The maximum bending moment is $R_M M_{AV} = 1{\cdot}42 \times 54{,}000 = 76{,}700$ in.-lb. Therefore there is sufficient compressive resistance.

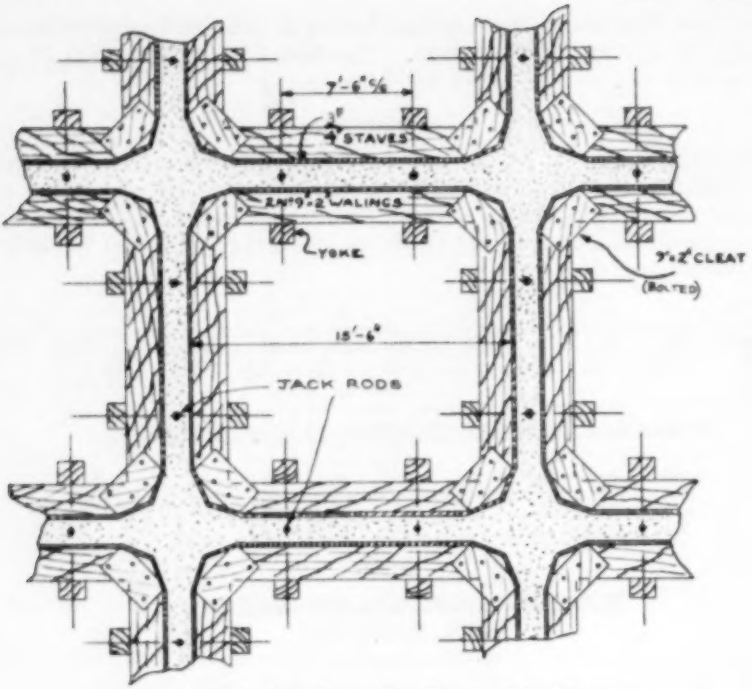


Fig. 2.—Wooden Form for Square Bin (Plan).

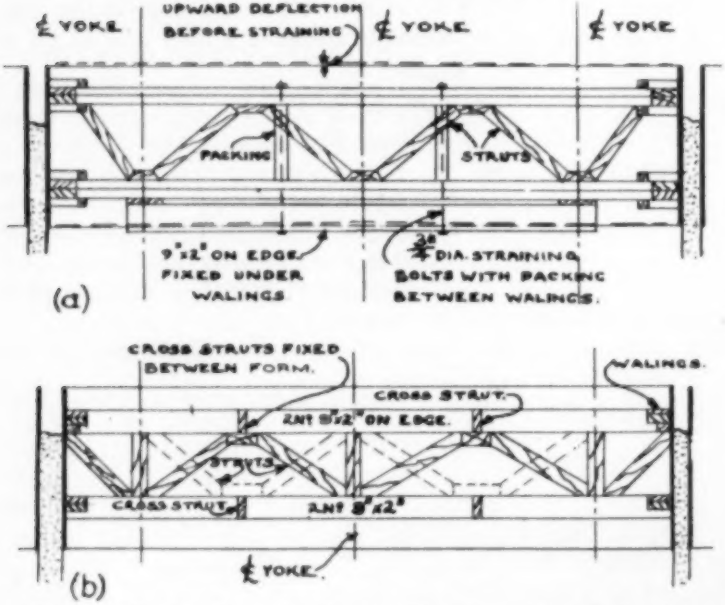


Fig. 3.—Wooden Form for Square Bin (Elevation and Section).

(*See facing page.*)

Construction with Moving Forms.—II*

BY L. E. HUNTER, M.Sc., A.M.Inst.C.E.

Construction of Wooden Forms.

THE forms can be either entirely of timber or partly of timber and partly of steel. If of timber they are built up of softwood walings, but the laggings or staves should be of good quality timber such as British Columbian pine free from knots and cracks. It is usual to waterproof the staves, and for this purpose linseed oil is best and also improves the appearance of the wall.

Fig. 2 (see p. 120) shows a form for a square bin. The construction is similar to that of a girder, and the elevation in *Fig.* 3 (*a*) (p. 120) shows staves attached to horizontal walings which are braced vertically by inclined posts bearing against

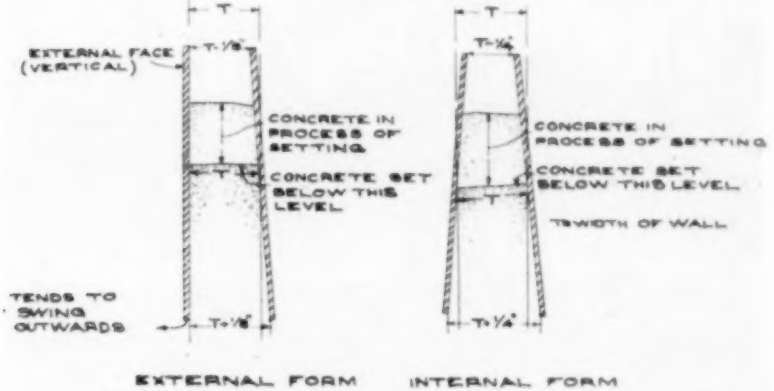


Fig. 4.—Showing Batter required on Wooden Forms.

chocks at both top and bottom. To ensure that the upward pull from the yokes is spread uniformly over the form and so obviate the possibility of the two walings separating, the walings are connected by straining bolts at 5-ft. or 6-ft. intervals. Thus, when the yokes move upward, the whole of the form moves with them and the tendency of the walings to buckle is reduced. The dimensions of the timbers are typical and can be used for the construction of a bin up to about 16 ft. wide.

There is no difficulty in providing the required rigidity of moving forms for short walls, but in the case of long walls more than the usual waling is required. *Fig.* 3 (*a*) shows the usual form in which the straining bolts are made longer than in the case of an ordinary shutter. Under the lower waling is a timber on edge of sufficient strength to provide the necessary support to prevent sagging of the walings, and consequently of the form. The stiffening timber is fixed by the straining bolts, and an initial upward deflection of the form can be caused by tightening the bolts. Sagging of a moving form is a serious matter as it can twist the forms for adjacent short cross-walls, the latter forms tending to come away from the wall at the top and consequently cause jamming against the hardened concrete at the bottom of the form.

* Continued from March, 1950.

The forms should be made and assembled in a joiner's shop, since they are too complicated to be made on an open site. The forms must be self-clearing, and for this reason they are provided with a small batter, except in the case of the external form on an outer wall.

Fig. 4 shows the amount of batter required on a form 4 ft. high ; the total batter is $\frac{1}{4}$ in. on each face, but may be $\frac{3}{4}$ in. especially when the temperature is high. The reason for the external form for an outside wall being vertical is the tendency for the upward pull on the form to result in a moment about the jack-

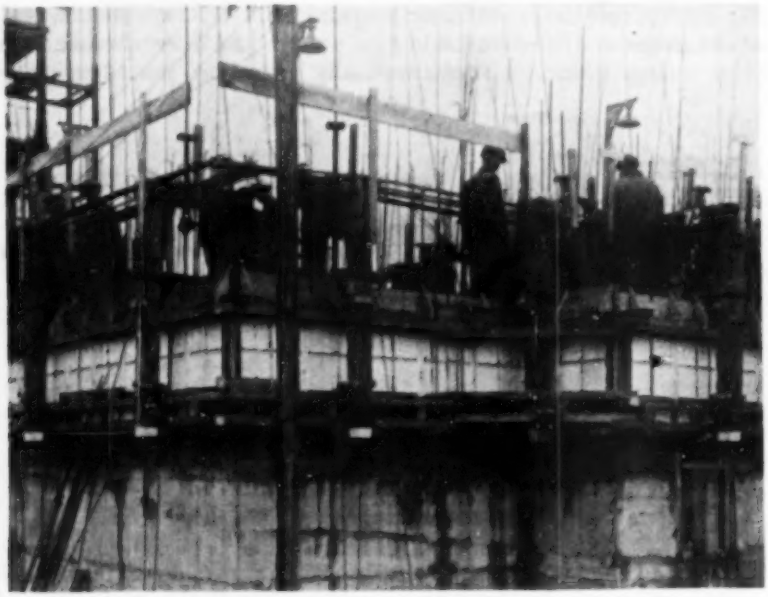


Fig. 5.—Steel Moving Forms for Rectangular Bins.

rod. On an inner wall the moments on each face balance each other, but on the outside form of an external wall the moment is not balanced and the result is that the outside form tends to swing outwards at the bottom.

It is a matter of opinion whether or not the form should be 4 ft. deep. It is a very safe depth to adopt for construction in the British climate. The initial setting time of the cement is the criterion and, in climates where the heat is such that the initial set is considerably accelerated, a form 3 ft. deep could be used and a saving achieved. As the initial setting time of ordinary and rapid-hardening Portland cements is about the same, there is no advantage in using rapid-hardening Portland cement in moving-form construction ; ordinary Portland cement is recommended for this method.

Steel Forms.

If steel forms are used instead of timber staves, proprietary panels are generally used. Steel forms have high salvage value and can be used many

times. An example of this type of form with timber walings is shown in *Fig.* 5 in use for rectangular bins. A suitable type for circular bins is that in which the forms are, say, 4 ft. deep and, say, 5 ft. long, made up of plates 5 ft. long by 8 in. wide, stiffened by 2-in. by 2-in. by $\frac{1}{4}$-in. angles at the top and bottom, a central horizontal 3-in. by 3-in. by $\frac{11}{16}$-in. tee, and diagonals of 2-in. by 2-in. by $\frac{1}{4}$-in. angles. It is essential to weld the angles and tee to the plate to the required curvature. The angles and the tee are usually previously bent to a template. The ordinary form of yoke used for timber forms, as described later, can be adopted for steel forms. If the start of the moving is at a con-

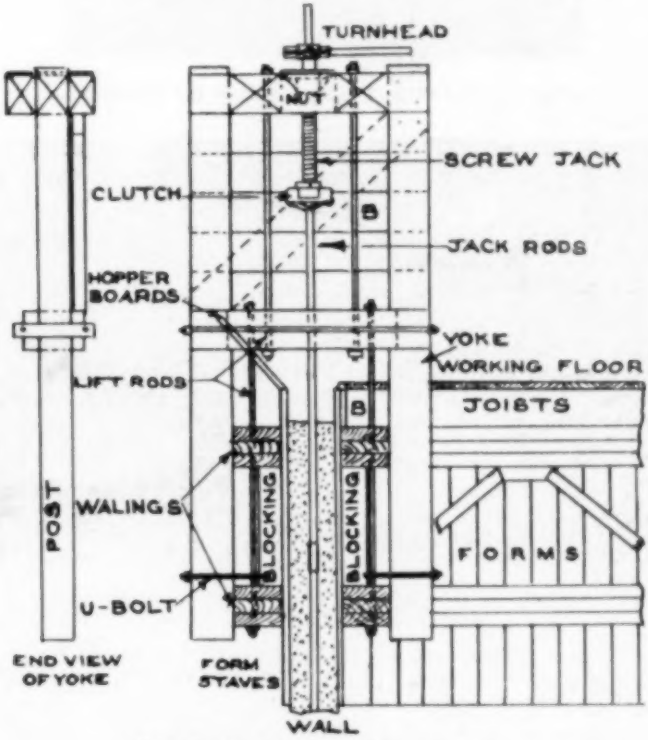


Fig. 6.—Details of Yoke and Jack.

siderable height above the ground, it is preferable to hoist the plates on to a platform at starting level, and there assemble and weld the forms. The dismantling of steel forms must be carefully considered when constructing them. As the parts are welded it is impossible to break the joints satisfactorily, especially as this work takes place at the top of the structure where the roof or slab over the bins is concreted with the moving forms and deck still in position. Dismantling will generally be easy if the angles and tees are cut at the joints in the plate. Small holes in the tees permit steel cables to be attached for lowering the parts. A better, but more costly, procedure is to weld two eyebolts to the top angle

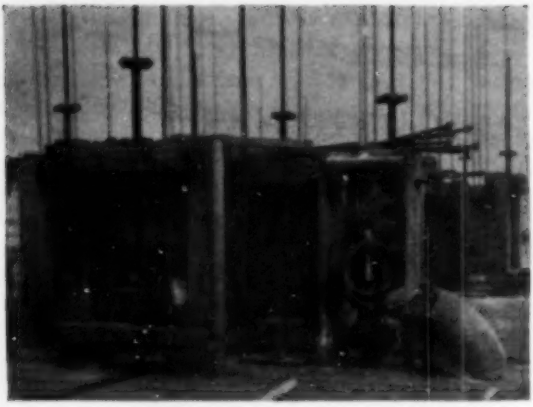


Fig. 7.—Yokes in Position Ready for Moving.

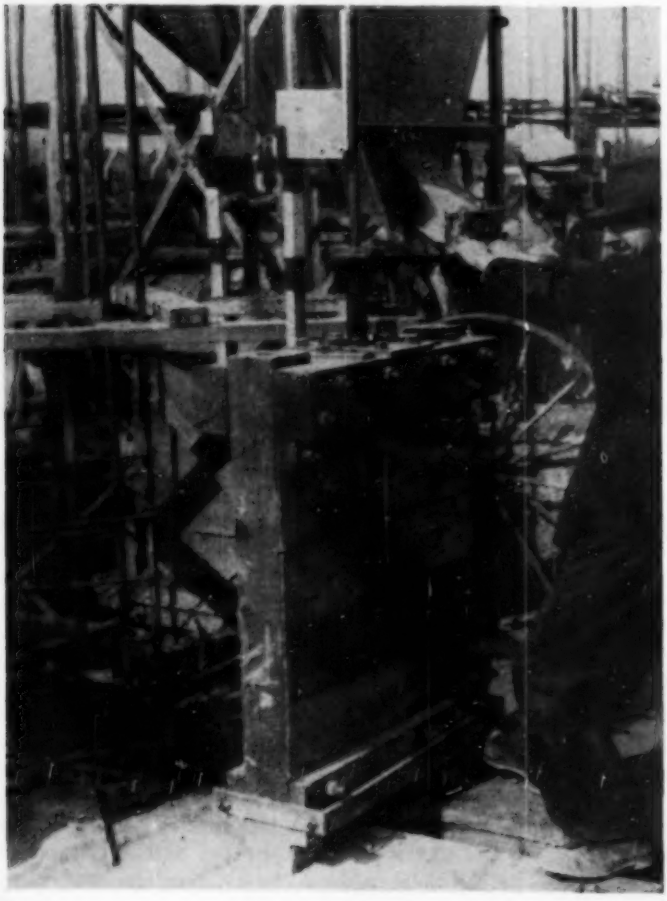


Fig. 8.—Jacking in Progress.

on each plate. The steel cables are fixed to the eyebolts before the joints are cut, and when this is done the plates can be lowered to the ground.

Yokes and Lifting Equipment.

The yokes to which the forms are attached, and which are suspended from the steel jack-rods embedded in the concrete, comprise two vertical legs connected together by timber transoms (*Fig.* 6) to which the screw-jack is attached. The screw-jack on the jack-rods raises the yoke which lifts the forms by means of two lift-rods fixed to the bottom transom of the yoke and the two walings of the forms as seen in *Fig.* 6. The overall height of the yoke is about 6 ft., the

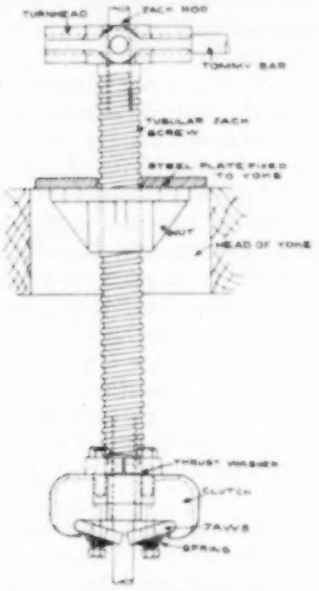


Fig. 9.—Details of Lifting Jack.

timbers comprising the legs being about 6 in. square. It is frequently necessary slightly to recess the walings to allow the legs to pass, but there is no actual connection between the legs and the walings. The legs form a guide for the forms and ensure that the yoke keeps vertical. The upper and lower transoms are connected by two steel tie-rods. The back of the yoke between the transoms is covered with boards across which a diagonal brace is fixed, thereby ensuring that the yoke does not warp. All the parts are well bolted together. Details of yokes are also seen in *Figs.* 7 and 8.

The jacks are of special design, and one type is shown in *Fig.* 9. The tubular screw is 2 in. diameter with $\frac{1}{2}$-in. pitch, and the central hole is about $1\frac{1}{8}$ in. diameter. A clutch, which comprises a pair of hinged and serrated tool-steel jaws, and is capable of being rotated, is provided at the lower end of the screw. The jaws bite into the jack-rod under the action of conical springs resisting downward movement of the jaws, which are free to move upwards but for the interposition

of the jack-rod. A jack is shown being operated in *Fig.* 8. The jaws of jacks are a frequent source of weakness, and it is essential that they be of good-quality steel and thick enough to withstand the extremely heavy load placed upon them when the shutters stick or when the forms grip the concrete due to the forms being out of alignment. Due to the slight variability of the diameter of the jack-rods it has been found that the jaw type of clutch may be uncertain and may break, and consequently an improved type of grip is a sliding sleeve fitted with a self-centering ball bearing.

The jack-rods are generally 1 in. or 1⅛ in. diameter, and are of medium tensile steel. The longest jack-rod used is that which can be easily handled, and this is 16 ft. to 18 ft. In moving-form construction it is essential to remember that much of the work is undertaken at a high level where wind may be troublesome, and for this reason all parts should be reduced to a size which can be handled without danger. Jack-rods are not all of the same length, since it is necessary to ensure that all the joints in the rods do not occur in the same horizontal plane. For this reason the rods in the lowest set are of different lengths and vary by multiples of 2 ft. from, say, 12 ft. to 18 ft. Another advantage of staggering the joints is that the labour of placing subsequent rods is spread throughout the whole time of moving and one or two men suffice for this operation. If the joints in the rods were all at the same level it would be necessary to employ more men and the moving would be stopped for a time. Also, the effect of the joints in all the rods being at one level would be highly dangerous to the equilibrium of the forms for jointing the rods. Short lengths of steel tube are used. They are about 6 in. long and are indented in the middle to provide a seating for the top of the lower rod. Sometimes the rods are joined by a screwed steel stud, which is preferable, since a plain tube does not provide rigidity for the bottom of the upper rod which, under load, behaves as a free-ended strut at the bottom, and if the rod should give under its load it is at this point that the rod generally bends.

The jack-rod in all cases behaves as a strut and, consequently, the rigidity of the rod imposes a limit on the load the yoke can sustain. Thus, the more rigid the connection between the individual rods the better, although it is rarely that the unsupported length of a jack-rod exceeds 4 ft. No support is obtained, however, from the bottom transom of the yoke. The jack-rods may easily be buckled during moving. The worst condition of the rods is when the work is started, as at this stage the rods are encased in concrete which is only a few hours old and are not gripped by the set concrete. It is an advantage to provide starter rods fixed in the set concrete and projecting from 2 ft. to 6 ft. above the level of the concrete in the fixed shutters. This is a little more expensive, but reduces the risk of the rods buckling due to premature lifting. If possible, it is best for the jack-rods to be embedded in the foundation or other construction below that for which the moving-forms are used. Where a jack-rod passes through an opening in a wall it is necessary to support it, and sometimes it may be necessary to support the rods through the whole of a floor. This is done by bracing the rods at intervals of 2 ft. or so with timbers. Details of such cases will be given in a later article.

(*To be continued.*)

Steel-alloy Bars used in Prestressed Concrete.

As a result of much research and many tests, a method of prestressing concrete members by using bars of high-tensile steel alloy, not previously available for structural work, has been developed. New types of end anchorages and a simplified method of jacking are features of the system. The special bars will shortly be available in diameters up to 1¼ in. with threaded ends and anchorages (*Fig.* 1).

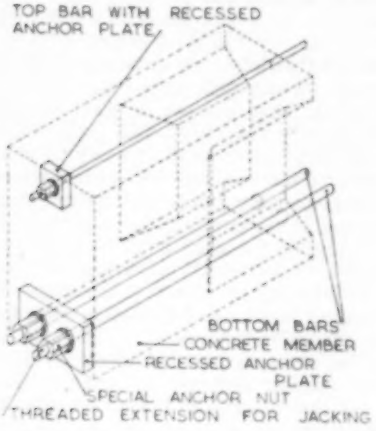


Fig. 1.—Anchorages.

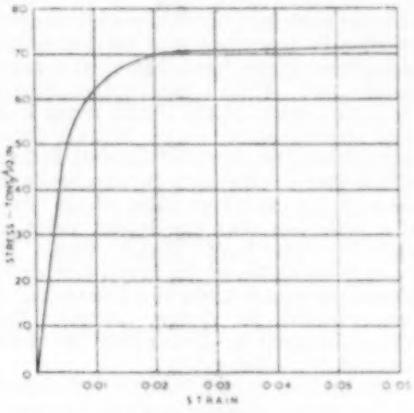


Fig. 2.—Stress-Strain Curve for Steel-alloy Bars.

Due to the large diameter of the bars and because the steel is fairly resistant to corrosion, the system is particularly suitable for prestressed concrete construction where durability is essential.

The special bars have a tensile strength of 70 tons per square inch, and a 0·1 per cent. proof stress of over 54 tons per square inch. There is no marked yield point (*Fig.* 2) but the steel is re-

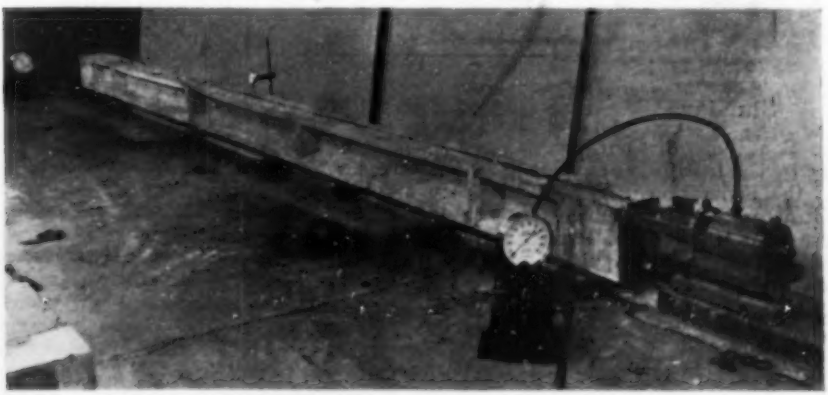


Fig. 3.—Prestressed Concrete Test Beam and Jack.

latively ductile, having an elongation on an 8-in. gauge-length of 8 per cent. or slightly more. The bars must not be hot-worked or welded. The initial tension recommended is 42 tons per square inch and, as the creep of the steel is insignificant at this stress, the reduction of initial strain is almost entirely due to the contraction of the concrete. The reduction of initial tension in the alloy bars stressed initially to 42 tons per square inch is slightly less than the corresponding reduction in the hard-drawn wire commonly used. If the reduction is expressed as a proportion of the initial stress it is only slightly more, partly because there is practically no creep in the steel and no initial slip in the end anchorages, and because the elastic modulus of the alloy is a little less than the lowest value of plain carbon hard drawn wire. An average reduction of the initial tension of 7 tons per square inch, or 16 per cent., is recommended for design, although the reduction may be less under specially controlled conditions.

Special nuts, which have been designed for use on the threaded ends to the bars, provide an anchorage practically equal in value to the full strength of the bar. Reduction of initial tension by slipping of the anchorages is avoided because the nuts are tightened before releasing the pull of the jack.

The jack (*Fig.* 3) is of a new type and applies a direct pull of 35 tons to 42 tons. A threaded adapter to suit the diameter of the bar is secured to the jack by a cotter, and the adapter grips the bar by being threaded thereon. This method of stretching the bars has been found to be simple and quick, and the jack is small enough to be readily moved about on the rollers on which it is mounted.

Tests.

Test beams made at the Field Test Unit of the Ministry of Works include beams of I and U section and of spans from 20 ft. for a 12-in. by 9-in. I-beam to about 50 ft. for a 30-in. by 13½-in. I-beam. All the tests gave satisfactory results and in no case did an end-anchorage fail. Some of the beams were precast in short lengths subsequently assembled and prestressed. The holes to accommodate the bars were formed in the concrete by rubber tubes, and the bars were grouted after stretching. Grouting is recommended for protection of the bars in foundations or for members exposed to industrial atmospheres, but otherwise is not necessary. For example, beams can be made of U section with the bars exposed in the trough. The grouting may be postponed and the reduction of initial tension reduced by re-stretching the bar just before the beams are erected in position. The jacks are sufficiently simple and portable to make this procedure worth while in large or important structures.

The development of the system has been carried out by Mr. Donovan Lee, M.Inst.C.E., and the tests were made under the supervision of Professor A. D. Ross, M.Inst.C.E. It is understood that the steel components and jacks for this system will be available from Messrs. McCall & Co. (Sheffield), Ltd., who have been associated with Mr. Lee in the development.

Standard Form of Contract for Civil Engineering Works.

THE Conditions of Contract prepared in 1945 by the Institution of Civil Engineers and the Federation of Civil Engineering Contractors and recommended by both these bodies for use in connection with civil engineering construction, has now been revised in collaboration with the Association of Consulting Engineers.

New clauses in the 1950 edition or alterations to clauses in the 1945 edition relate to adverse physical conditions and artificial obstructions, care of the works, excepted risks, damage to persons and properties, giving notices and paying fees, extraordinary traffic, wages and conditions of labour, insurance of works, costs of samples and tests, maintenance, provisional and prime-cost sums, monthly payments, frustration, and settlement of disputes. A revised clause concerning variation of the price of labour and materials is also included.

Design of Reinforced Concrete Members in accordance with the British Standard Code*—III.

By CHAS. E. REYNOLDS, B.Sc., A.M.Inst.C.E.

Columns Subjected to Axial Loads.

THE accompanying *Tables* Nos. 5 and 6 apply to the design of short axially-loaded columns, the permissible compressive stresses c_1 in 1 : 2 : 4, 1 : 1½ : 3, and 1 : 1 : 2 concrete being 760 lb., 950 lb., and 1140 lb. per square inch. Methods of using the tables for other conditions are described subsequently. The units are inches and pounds throughout.

When calculating the safe load on a column subjected to an axial load, the stress t_c in the main reinforcement is independent of the stress in the concrete, and is 18,000 lb. per square inch in mild steel. It is convenient to express the load-carrying capacity of a column as the product of the intensity of load K_1 and the cross-sectional area A. The value of K_1 depends on the maximum compressive stress c_1 permitted for the mixture of the concrete used and the percentage $p\left(=\frac{100A_c}{A}\right)$ of main reinforcement, and is $c_1 + 0{\cdot}01p(t_c - c_1)$ lb. per square inch (if c_1 and t_c are in lb. per square inch). Values of K_1 for the three mixtures described in the Code and for p from 0·8 per cent. (minimum) to 8 per cent. (maximum) are given by the curves in *Table* No. 5, which also give particulars of the cover of concrete and sizes of bars permitted. The upper part of this table applies to short columns with lateral ties, and *Table* No. 6 gives the safe loads on similar square columns of various sizes.

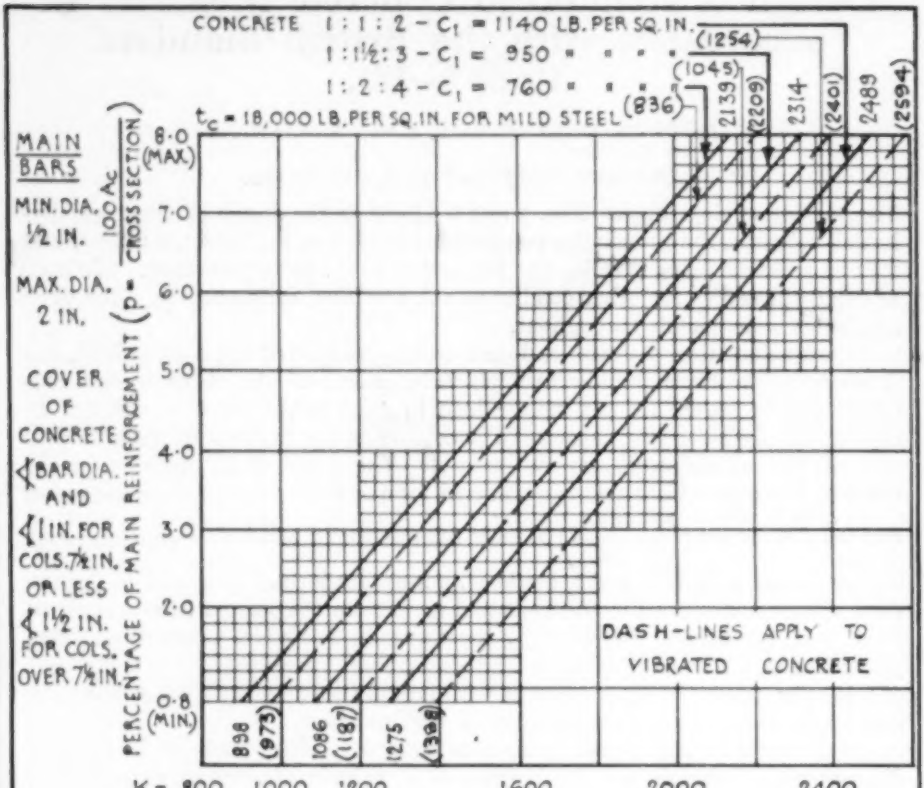
The size of a short square column to carry a specified load can be selected directly from *Table* No. 6. To design a short column of any other shape than square, it is necessary only to assume a size, divide the total load by the cross-sectional area, and then determine the percentage of reinforcement from *Table* No. 5 by reference to the curve for the particular mixture of concrete. The nature of the problem will determine whether the column must be the most economical (in which case the richest mixture and the smallest percentage of reinforcement generally apply), or whether it is to have the smallest outside dimensions (that is the richest mixture and largest percentage of reinforcement apply), or any intermediate conditions. The size may be specified to enable certain shuttering to be used or to conform to architectural requirements, or the mixture of the concrete may be specified to avoid using a mixture different from that in adjacent floors or other work. If one or more factors are so specified the number of suitable columns is less than if the design is unrestricted.

Several factors may necessitate variations from the foregoing methods of calculating the safe axial load on columns.

VIBRATED CONCRETE.—The increase of 10 per cent. in the permissible stress in vibrated concrete can be allowed for by using the data in *Table* No. 5. The safe intensity of load on a short column of any shape with lateral ties is

* Continued from January and February, 1950.

TABLE 5.—INTENSITY OF LOAD ON SHORT COLUMNS.



INTENSITY OF AXIAL LOAD = $K_1 = C_1 + 0{\cdot}01p(t_c - C_1)$ LB. PER SQ. IN.

$$K_1 = \frac{\text{TOTAL LOAD}}{\text{CROSS SECTIONAL AREA}}$$

CROSS-SECTIONAL AREA = AREA OF CROSS SECTION FOR COLUMNS WITH LATERAL TIES.
OR = AREA OF CORE FOR COLUMNS WITH HELICAL BINDING.

ADDITIONAL LOAD DUE TO HELICAL BINDING = $k_2 D_c$

PITCH OF HELICAL BINDING IN.	VALUES OF K_2 — DIAMETER OF HELICAL BINDING (IN.)									D_c MIN. IN.
	3/16	¼	5/16	⅜	½	5/8	¾	7/8	1	
1	2,340	4,140	6,510	–	–	–	–	–	–	6
1½	1,560	2,760	4,350	6,250	11,080	–	–	–	–	9
2	1,170	2,070	3,265	4,700	8,300	13,000	–	–	–	12
2½	936	1,660	2,610	3,750	6,640	10,400	15,000	–	–	15
3	780	1,380	2,170	3,120	5,540	8,670	12,500	17,000	22,200	18

$K_2 = 0{\cdot}523\,A_B\,t$

t = 13,500 LB. PER SQ. IN.

A_B = VOL. OF HELICAL BINDING PER UNIT LENGTH OF COLUMN

D_c MIN. = 6 × PITCH

K_1, the value of K_1 being that obtained from the dash-lines in *Table* No. 5; or by direct calculation the safe load is $1{\cdot}1c_1(A - A_c) + 18{,}000A_c$.

COLUMNS WITH HELICAL BINDING.—The rules in the British Standard code for columns with helically-bound cores are the same as those in the D.S.I.R. code but with higher stresses in the concrete and main reinforcement; the stress (13,500 lb. per square inch) in the helical binding is, however, the same. Therefore the load carried by the concrete and main reinforcement can be evaluated from *Table* No. 5, observing that the stressed area in this case is the cross-sectional area of the cylindrical core within the binding. The additional safe load due to the helical binding is the same as in the D.S.I.R. code and, although given elsewhere, is also given at the foot of *Table* No. 5 for completeness.

WIND STRESSES.—Since the increase of 33⅓ per cent. applies to the stresses in the reinforcement and concrete, the safe axial loads on short columns with lateral ties or helical binding given in *Tables* Nos. 5 and 6 can be increased by one-third.

LONG COLUMNS.—A short column is one in which the ratio of the effective length to the least lateral dimension does not exceed 15. If this ratio is exceeded (up to a maximum permissible ratio of 45) the safe load calculated in accordance with the foregoing must be reduced by the same proportions as are specified in the D.S.I.R. code and the London County Council By-laws (1938).

EXAMPLES OF DESIGN OF AXIALLY-LOADED COLUMNS.

In the following examples a column is designed to carry an inclusive load of 400,000 lb. in conformity with each of the conditions stated. Mild steel reinforcement is used throughout. *Tables* Nos. 5 and 6 are used, and also *Table* No. 1 which gives the cross-sectional areas of round bars and which was included in Part I (January, 1950).

(*a*) A square column of 1 : 2 : 4 ordinary concrete with lateral ties. From *Table* No. 6, suitable columns are 18 in. square with eight 1¼-in. bars; 20 in. square with four 1⅜-in. bars or eight 1-in. bars; and 21 in. square with four 1⅛-in. bars or eight ⅞-in. bars.

(*b*) The smallest square column with lateral ties (vibrated concrete). For this condition the richest concrete (1 : 1 : 2) and the largest percentage of reinforcement (8 per cent.) should be used, and the greatest permissible intensity of load K_1 (*Table* No. 5) is 2594 lb. per square inch. The gross cross-sectional area is therefore $\frac{400{,}000}{2594} = 154$ sq. in., and the size of the column is $\sqrt{154} = 12{\cdot}4$ in., say, 12½ in. square. The area of main reinforcement may be slightly less than $8 \times 0{\cdot}01 \times 154 = 12{\cdot}3$ sq. in., say, eight 1⅜-in. bars (= 11·88 sq. in., *Table* No. 1).

(*c*) The most economical square column with lateral ties. Generally the least reinforcement (0·8 per cent.), the strongest concrete (1 : 1 : 2 vibrated concrete), and a square cross section produce the cheapest column. For this case K_1 (*Table* No. 5) is 1388 lb. per square inch. The gross cross-sectional area is $\frac{400{,}000}{1388} = 288$ sq. in. The size of the column is $\sqrt{288} = 17$ in. square. The area of main reinforcement is $0{\cdot}8 \times 0{\cdot}01 \times 288 = 2{\cdot}3$ sq. in., say, four ⅞-in. bars (*Table* No. 1).

TABLE 6.—SAFE AXIAL LOADS ON SHORT SQUARE COLUMNS.

MIXTURE OF CONCRETE	SIZE OF SQUARE COLUMN	DIAMETER OF MAIN REINFORCEMENT (MILD STEEL) BARS														
		FOUR (4) BARS								EIGHT (8) BARS						
		5/8"	3/4"	7/8"	1"	1 1/8"	1 1/4"	1 3/8"	1 1/2"	3/4"	7/8"	1"	1 1/8"	1 1/4"	1 3/8"	1 1/2"
1 : 2 : 4 C_1 = 760 LB. PER SQ. IN.	10 IN.	97	106	118	130									REINFORCEMENT EXCEEDS 8 PERCENT.		
	11 IN.	113	122	133	146											
	12 IN.	131	140	141	164											
	14 IN.		179	150	203	218	233	LOADS ARE GIVEN IN UNITS OF 1000 LB.								
	15 IN.			213	225	220	256									
	16 IN.			236	249	263	279									
	18 IN.				289	304	320	337	357	296	318	343	372	404	440	489
	20 IN.					373	389	406	426	365	387	412	441	473	508	548
	21 IN.					404	420	437	457	390	418	443	472	504	540	579
	24 IN.	REINFORCEMENT IS LESS THAN 0·8 PER CENT.									521	546	575	607	642	682
	27 IN.											662	691	723	755	798
	30 IN.												821	853	888	978
1 : 1½ : 3 C_1 = 950 LB. PER SQ. IN.	10 IN.	116	125	136	149									REINFORCEMENT EXCEEDS 8 PERCENT.		
	11 IN.	136	145	156	169											
	12 IN.	158	167	178	190											
	14 IN.		216	227	240	253	270	LOADS ARE GIVEN IN UNITS OF 1000 LB.								
	15 IN.			245	257	272	288									
	16 IN.			274	287	301	317									
	18 IN.				361	376	392	409	428	368	390	415	444	475	510	549
	20 IN.					448	464	481	500	440	462	487	516	548	582	621
	21 IN.					487	503	520	539	479	501	526	555	587	621	643
	24 IN.	REINFORCEMENT IS LESS THAN 0·8 PER CENT.									629	654	683	715	749	788
	27 IN.											800	828	860	895	936
	30 IN.												991	1023	1057	1096
1 : 1 : 2 C_1 = 1140 LB. PER SQ. IN.	10 IN.	135	144	155	167									REINFORCEMENT EXCEEDS 8 PERCENT.		
	11 IN.	159	168	179	191											
	12 IN.	185	194	205	217											
	14 IN.		253	264	276	291	306	LOADS ARE GIVEN IN UNITS OF 1000 LB.								
	15 IN.			297	309	324	339									
	16 IN.			336	341	359	375									
	18 IN.				422	436	452	469	488	429	440	475	503	535	569	608
	20 IN.					513	529	546	565	506	527	552	580	612	646	685
	21 IN.					459	575	592	611	551	573	598	626	657	692	730
	24 IN.	REINFORCEMENT IS LESS THAN 0·8 PER CENT.									738	763	791	822	857	895
	27 IN.											927	955	987	1021	1060
	30 IN.												1160	1192	1226	1265

SHORT SQUARE COLUMN WITH LATERAL TIES:—
SAFE AXIAL LOAD = $c_1 D^2 + A_c(t_c - c_1)$ LB.

D = SIDE OF COLUMN (IN.)
A_c = AREA OF MAIN BARS (SQ. IN.)
t_c = 18,000 LB. PER SQ. IN. FOR MILD STEEL

(*d*) A column 16 in. by 20 in. with lateral ties; 1 : 2 : 4 ordinary concrete. $K_1 = \frac{400{,}000}{16 \times 20} = 1250$ lb. per square inch. From *Table* No. 5 the percentage of reinforcement p is 2·8. The area A_c of main reinforcement required is $2{\cdot}8 \times 0{\cdot}01 \times 16 \times 20 = 8{\cdot}85$ sq. in., say, six $1\frac{3}{8}$-in. bars (*Table* No. 1).

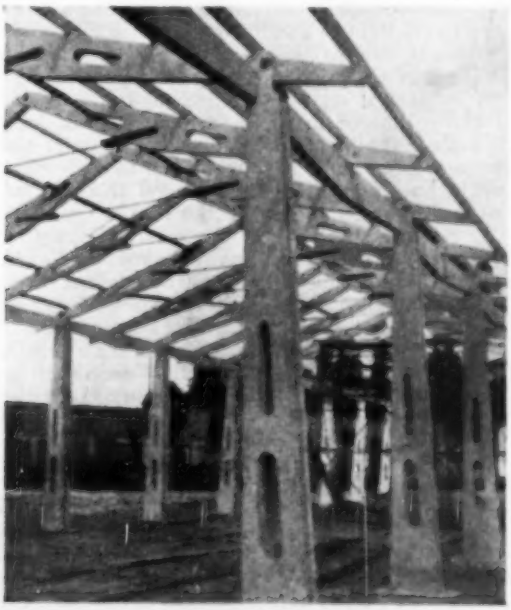
(*e*) An octagonal column, 18 in. across the flats; 1 : $1\frac{1}{2}$: 3 vibrated concrete; helical binding.

The diameter D_c of the core is $18 - 3 = 15$ in. Provide $\frac{3}{8}$-in. helical binding at 2-in. pitch ($K_2 = 4700$ from *Table* No. 5). The load carried by the binding is $4700 \times 15 = 70{,}500$ lb. The load to be carried by the concrete in the core and the main reinforcement is $400{,}000 - 70{,}500 = 329{,}500$ lb. The cross-sectional area of the core is $0{\cdot}7854 \times 15^2 = 177$ sq. in. The value of K_1 required is $\frac{329{,}500}{177} = 1865$ lb. per square inch. From *Table* No. 5, $p = 5{\cdot}8$ per cent., and $A_c = 5{\cdot}8 \times 0{\cdot}01 \times 177 = 10{\cdot}2$ sq. in., say, eight $1\frac{1}{4}$-in. bars (*Table* No. 1).

(To be concluded.)

Precast Building Frames.

THE accompanying illustration from "Il Cemento" shows the framework of a single-story building designed by an Italian engineer, Sr. A. Morganti, and constructed entirely with precast reinforced concrete members. The bolted connections enable the building to be easily dismantled. The high-level tie-rods and the overhang of the rafters at the eaves will be noted; these reduce the bending moments on, and therefore the size of, the rafters.



Precast Building Frames.

The British Standard Code of Practice for Slabs.

THE following notes are abstracted from a new British Standard code of practice, "Suspended Concrete Floors and Roofs (including Stairs)" [CP. 114, 100 to 105 (1950), obtainable from the British Standards Institution (Price 6s.)]. The code deals with suspended concrete floors and roofs of solid slab, flat slab, ribbed, precast, and filler-joist construction, and is generally applicable to stairs for which recommendations for loading and effective span are made. Where forms of construction which are not provided for in the code are employed, the ultimate failing load is three times the working load; where calculation is not practicable, tests must show that they attain this standard. The materials, strengths, stresses, loads, bending-moment coefficients, and the like are, unless otherwise stated, in accordance with British Standard Code CP. 114 (1948), "Structural Use of Normal Reinforced Concrete in Buildings."

CONCENTRATED LOADS.—No simple method of calculating bending moments, other than in the cases in the following, have been evolved. The effects of loads parallel to supports, the reduction in bending moments due to continuity, and spread of loads at supports, are matters requiring individual treatment.

If a slab is simply supported on two opposite edges and carries one or more concentrated loads in a line in the direction of the span, the bending moment may be assumed to be resisted by a width e of slab (measured parallel to the supports) equal to the following: (i) For solid slabs, e may be the sum of the load width, g, and $2{\cdot}4x(1 - x)$ ft., where x is the ratio of the distance from the nearer support to the section under consideration to the span. (ii) For slabs not included in (i), unless otherwise provided for in a sub-code, e depends on the ratio of the transverse and longitudinal flexural rigidities of the slab. When the ratio is about one, e is as for solid slabs, but as the ratio decreases a smaller value should be taken. The minimum value which need be taken is g plus $1{\cdot}2x(1 - x)$ ft., so that for a section at mid-span, e is $g + 3$ ft. (iii) Where the concentrated load is near an unsupported edge of a slab, e should not exceed the value in (i) or (ii) as appropriate, or half that value plus the distance of the centre of the load from the unsupported edge. The foregoing rules apply to bending moments only and not to shearing forces.

For slabs constructed of precast concrete members laid side by side, the effective width of distribution of concentrated loads is g plus the width of three precast members and joints if there is no topping, or plus the width of four members and joints if there is a topping at least 1½ in. thick.

PARTITIONS.—When the positions of partitions are known their weights should be included in the dead load, and they should be treated as concentrated loads. When the positions are not known the requirements of the Code of Functional Requirements of Buildings, Chapter V, "Loading", apply. Where floor slabs are not capable of effective lateral distribution, the designer should state this fact in writing to the responsible person concerned. Precast floors are not considered capable of providing effective lateral distribution of partition loads.

STAIR LOADING.—In the case of stairs with open wells, where spans partly crossing at right angles occur, the load on areas common to any two such spans may be taken as one-half in each direction. Where flights of steps or landings are built a distance of not less than 4½ in. into walls, and span in the direction of the flight, a 6-in strip may be deducted from the loaded area and the effective breadth of the section increased by 3 in.

EFFECTIVE SPAN OF STAIRS.—The effective spans of stairs without stringer beams should be taken as the following horizontal distances: (i) Where supported at the top and bottom risers by beams spanning parallel to the risers, the distance from centre to centre of beams. (ii) Where spanning on to the edge of a landing slab, itself spanning parallel to the risers, a distance equal to the "going" of the stairs plus at each end either half the width of the landing or 3 ft. whichever is the smaller. (iii) Where the landing slab spans in the same direction as the stairs, they should be considered as acting together to form a single slab.

SLABS SPANNING IN ONE DIRECTION.—The bending moments in slabs spanning

in one direction may be calculated on the assumption that the slab is continuous over the supports and is capable of free rotation about them; or in the case of uniformly-distributed or other symmetrical load on three or more nearly equal spans the bending moments may be assumed to be the following proportions of those induced at the middle of a freely-supported span similarly loaded: near the middle of an end span and at the support next to the end support, $\frac{1}{10}$; at the middle of interior spans and at other interior supports, $\frac{1}{12}$.

TRIMMING OPENINGS.—Openings in floors or roofs should be trimmed by beams or reinforcement so that the design strength of the surrounding slab is not impaired by the opening, regard being paid to the possibility of diagonal cracks at the corners of openings.

DEFLECTION.—The deflection of a slab should be such that the strength of the structure is not impaired or the finishes damaged. In normal cases this condition is satisfied if the ratio of the span to the depth does not exceed $\frac{1}{35}$ for slabs simply supported and spanning in one direction; or $\frac{1}{40}$ for similar slabs if continuous; or $\frac{1}{12}$ for cantilevered slabs.

JOINTS.—The position of construction joints should be determined by the designer.

Expansion joints alone are not recommended to allow for the effects of thermal movements in roof slabs because of the difficulty of arranging them at sufficiently small distances to obviate risk of movement. Insulation should be provided above the structural slab to reduce thermal movement. In large roofs not supported by framing, expansion joints are recommended in addition to insulation, the spacing of such joints being preferably not more than 100 ft. and being adjusted in accordance with the efficacy of the insulation. To prevent adhesion and reduce friction between a roof slab and its supports, it is advisable to insert building paper or other separating medium between them. In floor slabs rectangular in plan, unless there are conditions imposing abnormal changes of temperature, expansion joints are not generally required.

Expansion joints should be provided in floors and roofs of large buildings where abrupt changes in shape occur and at places where slabs are weakened by large openings.

TRANSVERSE REINFORCEMENT IN SOLID SLABS.—A solid slab spanning in one direction should have transverse reinforcement at right angles to the main reinforcement, and the total cross-sectional area of the transverse reinforcement should be not less than one-tenth of the total cross-sectional area of the main tensile reinforcement. The spacing of the transverse reinforcement should be not more than four times the effective depth of the slab.

RIBBED CONSTRUCTION.—A sub-code deals with floor and roof slabs consisting of reinforced concrete ribs cast in situ between blocks, arranged so that the ribs span in one direction only or in two directions at right angles, with or without a concrete topping. The blocks may form part of the slab or may be removed after construction.

Blocks made of burnt clay, when required to contribute to the strength of the slab, should have a crushing strength of not less than 3000 lb. per square inch of net section when axially loaded in the direction in which they will act in the slab. The concrete should be mixed in proportions to produce a sound concrete block. The thickness of any part of a concrete block should be not less than $\frac{1}{2}$ in. Blocks may also be made of metal, plywood, asbestos, or other material.

When concentrated loads or partitions are placed between the ribs, the topping should be designed to carry these loads from rib to rib.

The bending resistance of hollow-block construction may be based upon the strength of the reinforced concrete ribs (and topping where used) and the blocks (if properly jointed) in structural combination. Alternatively the blocks may be neglected. Where hollow blocks composed of material other than concrete are regarded as contributing to the strength of the slab, the permissible compressive stress in the blocks should not exceed one-fifth of their crushing strength. The elastic modulus of the material of the block may be assumed to be the same as for concrete. If hollow blocks of burnt clay or concrete are considered to add to the strength of the slab, the thickness of one wall of the block may be added to the thickness of the rib when calculating the resistance to shear. Alternatively, the

walls of both the adjacent blocks may be taken into account if the shearing stress does not exceed the safe shearing stress of the material. Where the width of a rib varies the average width at the section may be used in computing shearing stresses.

The thickness of the topping should be not less than 1½ in. or one-twelfth of the clear distance between the ribs, whichever is the greater, if the slab comprises permanent blocks regarded as contributing to the strength of the construction and having a clear distance between the ribs not exceeding 18 in. In all other cases, the thickness of the topping should be not less than 2 in. or one-twelfth of the clear distance between the ribs, whichever is the greater.

The width of the rib should be not less than 2½ in. The depth should be not more than three times the width. The spacing should be not more than 3 ft. between centres.

At least half of the main tensile reinforcement in the ribs should be carried through at the bottom on to the bearing and anchored. Continuous slabs in which it is impracticable to provide sufficient reinforcement to develop the entire moment of resistance at the support may be considered to be simply supported, but it is desirable to provide some reinforcement over the support; such reinforcement should have a cross-sectional area of not less than one-quarter of that in the middle of the adjoining bays and should extend at least one-tenth of the clear span into the adjoining bays. Provided that the standard bond and compressive stresses are reduced by 40 per cent., the space between parallel bars in the ribs may be reduced to ½ in. or the diameter of the bar, whichever is greater. For hollow-tile slabs having slip-tiles not less than ⅛ in. thick under the ribs, a cover of ½ in. may be provided for the bars above the tiles.

In slabs where the blocks or formers are to be removed or are of inadequate lateral stiffness, bridging extending the entire depth of the rib should be provided in spans greater than 12 ft. at intervals not exceeding 12 ft.

Blocks should be so arranged that the joints in adjacent rows are staggered. Where the blocks contribute to the struc-

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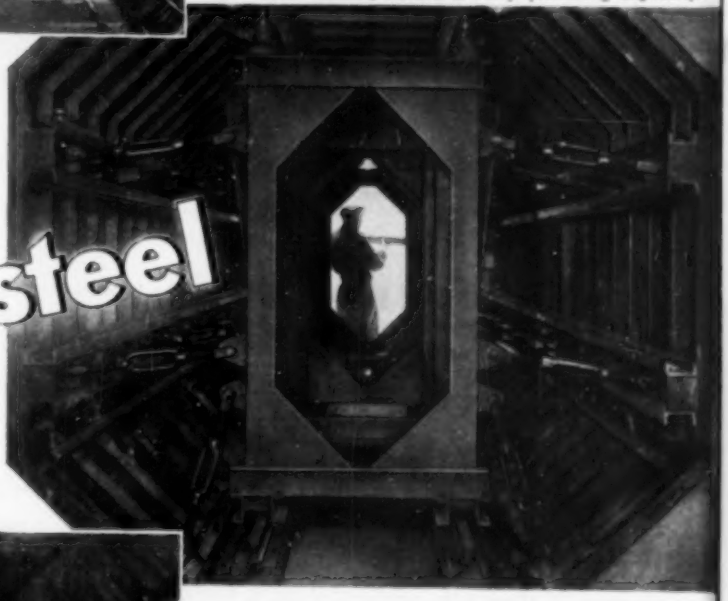

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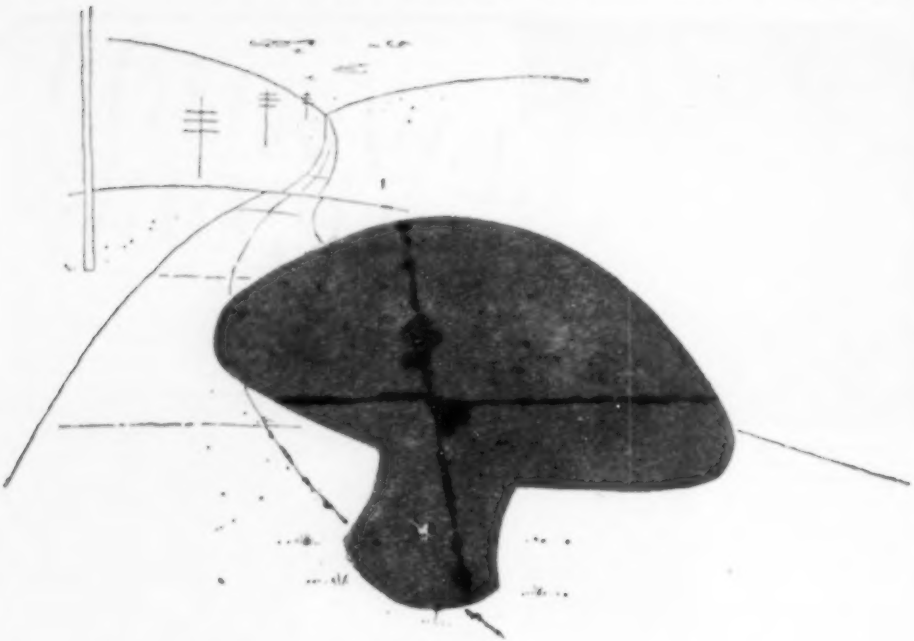

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tural strength of the slab they should be jointed with 1 : 3 cement and sand mortar.

Where a slab reinforced in one direction only is built into a wall or rests on a beam parallel to the ribs, a rib should be placed along the wall or beam, the minimum width of the rib being that of the bearing. The provision of reinforcement at right angles to the ribs should be considered. Where a slab butts against a wall parallel with the ribs, there should be a rib against the wall at least 2 in. wide.

Cooling Concrete in Dams.

In the spillway of the new Harlan County Dam, Nebraska, U.S.A., there will be 430,000 cu. yd. of plain concrete the specification of which requires that means must be taken to ensure that the temperature of the concrete shall not exceed 65 deg. F. or be less than 50 deg. F. To conform to this requirement the aggregate is cooled by circulating cold air through it. An air-conditioning plant cools the air to freezing-point, and the air is then blown up through the coarse limestone aggregate in the compartments of the batching plant. The materials in a 4-cu. yd. batch of concrete are 1128 lb. of cement, 3800 lb. of sand, 2300 lb. of ¾-in. stone, 2700 lb. of 1½-in. stone, 3300 lb. of 3-in. stone, 3200 lb. of 6-in. stone, 670 lb. of water, and 2¼ lb. of air-entraining agent. The temperature of the aggregate can be reduced by 35 deg. F. in 45 minutes if the temperature differential of the cooled air is 60 deg. F. The air is injected into the bins through a nozzle at the bottom.

In the Hungry Horse dam in Montana, pipes for circulating cooling water are being built into the structure. The dam, construction of which commenced recently, will contain nearly three million cubic yards of concrete and will be 564 ft. high. The total quantity of heat to be removed from the concrete is estimated to be equal to that liberated by burning 5500 tons of coal. About 5 million feet of thin-walled steel pipe will be embedded in the concrete.

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Insulation of District Heating Pipes with Cellular Concrete.

A SCHEME for the provision of hot-water central heating and hot-water supplies to a group of 1535 houses, shops, public buildings, etc., on a new housing scheme is described by Mr. R. J. D. Powdrill, Assistant Engineer of the Swindon Corporation, in a paper published in the Journal of the Institution of Municipal Engineers for November 1949. The scheme has since been curtailed to supply 134 houses only. The consulting engineers are Messrs. Donald Smith, Seymour, & Rooley, and Mr. J. Ackroyd is the Borough Engineer and Surveyor.

The method adopted of insulating the underground pipes between the electricity generating station, which is the source of the hot water, to the houses is shown in *Fig.* 1. The temperature of the water in the main flow is 180 deg. F., and in the return flow 140 deg. F. The pressure varies from 20 lb. to 50 lb. per square inch. Asbestos-cement pipes of 7-in. internal diameter and designed for an internal pressure of 260 lb. per square inch are being used.

The pipe ducts comprise a concrete base 4 in. thick recessed to receive lightly-reinforced precast concrete slabs 2 in. thick by 1 ft. high by 6 ft. long to form the sides. The joints are grouted with waterproofed cement-sand mortar. The vertical joints between the slabs have double rabbets, and these are also grouted with waterproofed cement-sand mortar. On poor ground and where the ducts pass under roads the concrete base and sides are made thicker. Generally the top of the duct is 18 in. below ground level. The ducts are filled with cellular concrete made with a foaming agent, and this is specified to have a thermal conductivity not exceeding 0·5 B.T.U. per inch of thickness per square foot per hour per

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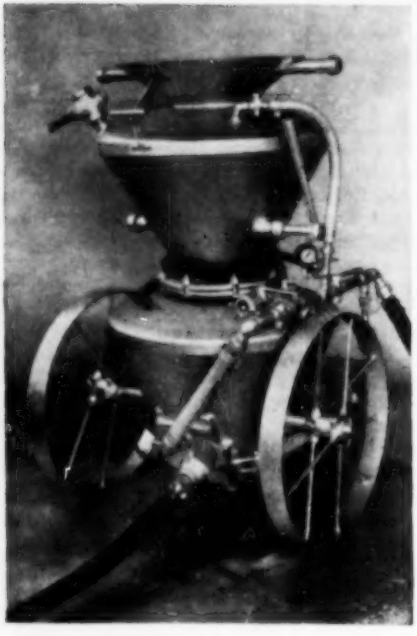

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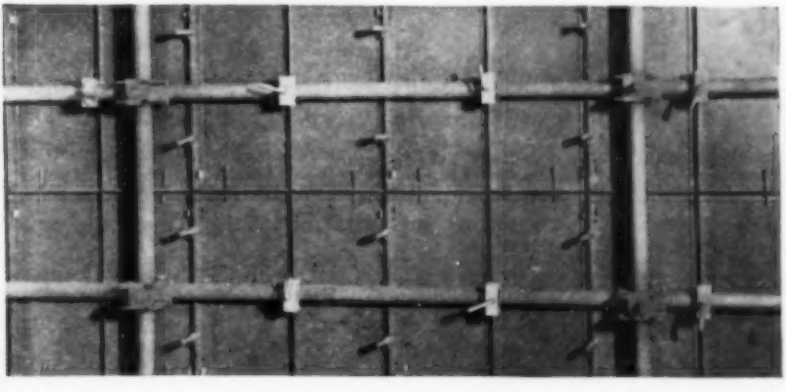

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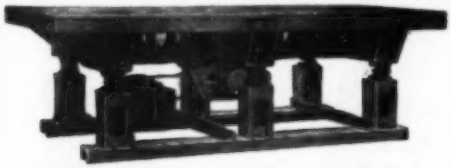

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degree Fahrenheit. The minimum compressive strength required is 45 lb. per square inch at 28 days. The density when dry is 20 lb. per cubic foot. The foam is produced in an aerating machine having an output of 9 cu. ft. of foam a minute. The slurry is mixed in a pan-type mixer in the proportions of 168 lb. of Portland cement to 60 lb. of water and the foam produced in 65 seconds, that is a little more than 9 cu. ft. At the sides

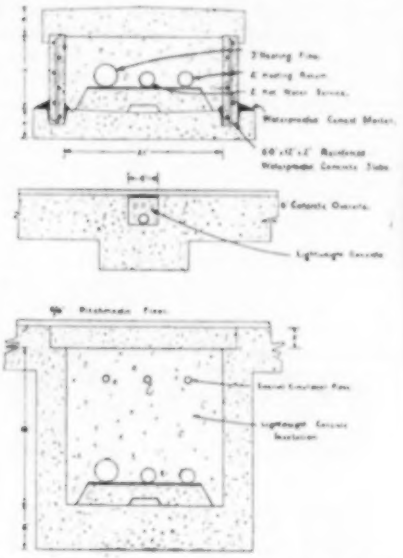

FIG. 1.—TOP: DUCT AS DESCRIBED; MIDDLE: CHASE IN HOUSE; BOTTOM: DUCT IN HOUSE.

and below the pipes the least thickness of insulation is 3 in., and over the pipes 4 in. The top of the duct is 4 in. thick and is placed *in situ*. The heat loss is from 12 to 18 per cent. of the heat transmitted.

Chemical Stabilization of Soil.

THE Massachusetts Institute of Technology is studying a new chemical process for the stabilization of soils, particularly with the idea of providing quickly a tough surface for the landing of aircraft and roads for military purposes. The treatment can be used on soils which contain as much as 30 per cent. of water by weight, and a few hours after treatment such soils are able to withstand heavy impact loads and to distribute static loads over an uneven foundation.

The process is based on the use of calcium acrylate, an organic chemical which is adsorbed by the soil particles, followed by sodium thiosulphate and ammonium persulphate which cause the calcium acrylate molecules and the soil particles to be locked together. The binding action begins two minutes or less after the mixing has taken place, and five hours after treatment the soil has a tensile strength of from 5 lb. to 10 lb. per square inch. If the stabilized soil is allowed to dry in air for a week or more, tensile strengths of 500 lb. per square inch or more are obtained, but this gain in strength is accompanied by a loss in elasticity. Rewetting of the dried soil causes a return to its original rubber-like consistency and its tensile strength is reduced. It is stated that a block of stabilized soil 3 in. thick withstood, without noticeable indentation, the weight of

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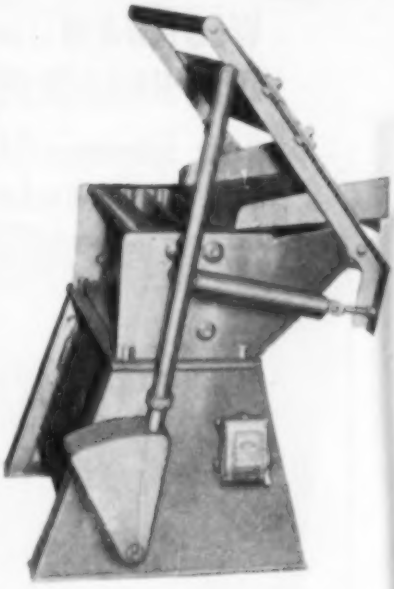

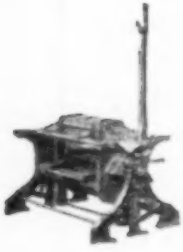

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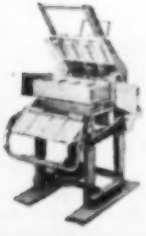

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a car moving over it 24 hours after treatment. A 16-lb. steel ball dropped on to the block from a height of 7 ft. rebounded about 6 in. without causing damage to the block.

Spacer for Reinforcement Bars.

A NEW German device (*Fig.* 1) for securing reinforcement bars in position is described in " Betonstein Zeitung " for November, 1949. It is a double ring of rust-resisting metal and is slipped on the bar by hand.

Fig. 1.—Spacer.

The inner ring grips the bar, and the outer ring maintains the distance between the bars or between a bar and the face of a concrete member. Both rings are formed from one piece of strip or wire. The spacer is sold by August Fechner und Sohn, Kleve (Rheinland).

A Film on Piling.

WE recently had an opportunity of seeing a film entitled " Basic Military Pile Driving " made for instructional purposes in the Royal Engineers. The film deals in a very practical manner with pile-driving equipment, setting-out piled foundations, and driving piles on land and in water. The piles dealt with are mainly timber piles and steel sheet-piles, but some reference is made to reinforced concrete piles and driving such piles by ordinary methods and with a water-jet. Although the structures shown are mainly military bridges, the film illustrates many hints and methods that should be of value to a civilian contractor engaged on the erection of temporary or permanent piled structures.

The film, which is in six parts (35 mm. or 16 mm.), is available for lending to engineering institutions and similar bodies, the charge being about 5*s.* for each part. Applications should be made to the Army Kinema Corporation, 36 Dover Street, London, W.1.

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Advertisements must reach this office by the 23rd of the month preceding publication.

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

BURSARIES IN CONCRETE TECHNOLOGY

NOTICE IS HEREBY GIVEN

that the election to Bursaries in Concrete Technology, tenable as from October 1950, will take place in June 1950.

Bursaries are of the value of £200 per annum, out of which the College Tuition Fee has to be paid. Under certain circumstances the amount will be increased (subject to the agreement of the donors).

Applications must be received on or before 31st May, 1950, by the DEPUTY REGISTRAR, CITY AND GUILDS COLLEGE, Exhibition Road, London, S.W.7, who will on written request send full information and application forms.

SITUATIONS VACANT

SITUATIONS VACANT. Consulting engineers, London, require experienced reinforced concrete detailers. Interesting work and prospects. Reply stating experience and salary required. BOX 2380, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

CIVIL ENGINEERING ASSISTANTS
(UNESTABLISHED)

Applications are invited from British-born subjects to fill the above vacancies with Ministry of Works at sites at Windscale, West Cumberland, Capenhurst (Chester); Harwell (Berks), and a site near Reading. Hostel accommodation available. Candidates should be capable of reading Building and Civil Engineering drawings and have knowledge of use of level and Theodolite for setting out and checking. Duties should prove of considerable value to men studying for Civil Engineering Qualification and needing site experience. Commencing salary up to £420 per annum, according to age and experience, on a range of pay rising to £495 per annum. Written applications giving details of age, education and previous appointments should be addressed to—

LONDON APPOINTMENTS OFFICE, MINISTRY OF LABOUR AND NATIONAL SERVICE, 1-6 Tavistock Square, W.C.1, quoting reference No. K.K. 92

within 14 days of the appearance of this advertisement. Copies of testimonials not required. Only candidates selected for interview will be advised.

SITUATION VACANT. We are requiring a man who is thoroughly experienced in the design of reinforced concrete structures. He should hold a University degree and/or A.M.Inst.C.E. The vacancy occurs in our Glasgow design office. Write in confidence to the Managing Director, BOX 2363, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Men who are experienced in the design of reinforced concrete structures and who hold a University degree and/or A.M.Inst.C.E. are invited to apply for a post we have vacant at our Manchester design office. Write, giving full details of experience and salary required, to the SECRETARY, BOX 2364, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Required for overseas work, engineer experienced in tunnelling in conglomerate rock. The applicant must be capable of pre-planning and estimating as well as supervising the actual work. Apply, giving full details, BOX 2378, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

(*Continued on page* 142.)

MISCELLANEOUS ADVERTISEMENTS

(Continued from page 141.)

SITUATION VACANT. Reinforced concrete designer with wide experience in all types of reinforced concrete structures required for project section of a London firm of reinforced concrete engineers. Salary up to £900 per annum. Write, giving full details of experience, training and age. Applications will be treated in confidence. Box 2373, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Inspecting engineer with practical experience of reinforced concrete construction and some design knowledge required by leading firm of reinforced concrete specialists to visit sites of work under construction, explain details of work to be carried out, and write concise reports. Box 2381, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience. Five-day week, staff pension scheme, social club and canteen. Apply to B.R.C. Engineering Co., Ltd., Stafford.

SITUATION VACANT. Contracts supervisor required to take charge of reinforced concrete and hollow tile floor contracts. Permanent position. Bonus and pension scheme. Write stating age, full particulars and salary required. Box 2379, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Experienced senior reinforced concrete designer required who is familiar with L.C.C. requirements and can undertake preliminary design and own production of detail drawings. Some knowledge of civil engineering and structural steelwork desirable. Varied interesting work in all engineering branches. London, West-End area. Good prospects and salary with additional incentive bonus. Advertiser's staff has been notified of this advertisement. Write, stating age, experience and salary required, to Box C.C. 626, at 191 Gresham House, London, E.C.2.

SITUATIONS VACANT. Reinforced Concrete Designers and Draughtsmen required at an early date for important works on several large Generating Stations. Positions are of a permanent character. A full-scale Superannuation Scheme is in operation. Applicants are asked to send details of experience and scale of salary required, and to state when available for interview and explanation of duties in Manchester, to C. S. Allott & Son, Consulting Civil Engineers, 1 North Parade, St. Mary's Parsonage, Manchester 3.

THE TRUSSED CONCRETE STEEL CO. LTD., REINFORCED CONCRETE ENGINEERS AND CONTRACTORS, invite applications for the following positions on their permanent staff.

(*a*) Engineers for the Design Office.
(*b*) Detailers and Draughtsmen for the Design Office.
(*c*) Engineers for the Construction Department.

Wide experience—all types of reinforced concrete work.

All positions are progressive and pensionable, and vacancies occur in London, Birmingham, Manchester and Glasgow. Five-day week. Salary according to experience and ability. Details of training, experience, and salary expected should be sent to Truscon House, 35/41, Lower Marsh, London, S.E.1 (adjacent to Waterloo Station). All replies will be treated as confidential.

THE LIVERSEDGE REINFORCED CONCRETE ENGINEERING CO. LTD.

require additional experienced Designers, Detailers and Draughtsmen. Permanent positions and five-day week. Applications in confidence, stating full details of experience, qualifications, age, salary required, to 31a John Adam Street, Adelphi, W.C.2.

SITUATIONS VACANT. Draughtsmen, draughtsmen-detailers and design engineers with experience of reinforced concrete structures are required by a large firm of reinforced concrete engineers in the Birmingham area. Write Box 2365, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. Designer-draughtsmen urgently required by Consulting Engineers for reinforced concrete work. Experience colliery structures an advantage but not essential. Apply by letter stating age, experience and salary required to J. C. Hughes & Partners, 119 Marylebone Road, London, N.W.1.

SITUATIONS VACANT. Reinforced concrete draughtsmen-detailers and designer-draughtsmen required in London Office of Consulting Engineer. Permanent positions. Excellent prospects. Write stating experience and salary required. Box 2386, "Concrete and Constructional Engineering," 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Technical sales representative, with wide contacts in building and civil engineering fields, required by a large modern firm of prestressed and precast concrete manufacturers. Experience in negotiations with consultants necessary. Apply, stating age, qualifications and terms expected. Box 2369, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Reinforced concrete designer-detailer, with not less than five years' designing experience, wanted for South Africa, East Africa, or Southern Rhodesia. Write stating age, marital condition, and experience. Salary up to £1,200 per annum, according to experience. Write Box R.C.D., c/o 95 Bishopsgate, London, E.C.2.

SITUATION VACANT. Designer-draughtsman for senior position. Must have considerable previous experience in the design of hollow tile floors and structural reinforced concrete design. Progressive permanent position. Pension scheme. Alternate Saturdays. Victoria Street district. Apply in strict confidence. Box 2382, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Civil engineer, age 25–28, required for site inspection work at head office of consulting engineer, Westminster. Applicants must have at least five years' experience on outside work with contractors. Good knowledge of soil mechanics an advantage but not necessarily essential. Salary £550–£800, according to age and experience. Box 2383, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience for their London office. Five-day week, staff pension scheme and canteen. Apply to B.R.C. Engineering Co., Ltd., Stafford.

SITUATIONS WANTED

SITUATION WANTED. Enterprising young engineer and precast concrete specialist requires progressive position. Experience includes design, development, and works and site control. Southern counties preferred. Box 2384, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

WORKS MANAGER with extensive technical and mechanical experience in spinning and other processes. G. C. M. Gould, formerly Major, R.E., shortly to sever his connection with Spun Concrete, Ltd., would be interested in active post with precast concrete manufacturers contemplating development on suitable lines. Some capital available. Write: High Bank, Playden, Rye, Sussex.

REINFORCED CONCRETE DESIGNER with experience is able to undertake part-time work at home for consulting engineers. Design and detailing of all types of reinforced concrete structures. Write, giving your requirements, to Box 2385, Concrete and Constructional Engineering, 14 Dartmouth Street, Westminster, S.W.1.

TESTING APPARATUS

MOISTURE TESTING. — Famous Engineers write about "SPEEDY" MOISTURE TESTER: "Not only 'speedy' but can be used satisfactorily by unskilled labour during tests." Accurate; portable; non-electric; inexpensive; used extensively for rapid determination of water-content of concrete aggregates, clays and soils. Write for full particulars to (Dept. CCE/4) Thos. Ashworth & Co., Ltd., Burnley, Lancs.

FOR SALE

FOR SALE. Steel piping. 5,000 feet of 3-inch, 5,000 ft. of 6-inch, and 2,000 ft. of 8-inch diameter. All screwed and socketted. Other sizes in steel piping available. Telephone: Mayfair (London) 5614 (4 lines).

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